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AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

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VOL. LXV

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CONTENTS

PAPERS

No.		PAGE
1118	**CAISSON DISEASE AND ITS PREVENTION.**	
	By **Henry Japp**	1
	Discussion:	
	By J. S. Haldane	24
	Frederick L. Keays	26
	Albert J. Loomis	29
	William L. Saunders	30
	Walton I. Aims	32
	Seward Erdman	32
	T. Kennard Thomson	33
	Henry Japp	34
1119	**COPYRIGHT IN DRAWINGS OF A TECHNICAL CHARACTER.**	
	By **D. A. Usina**	38
1120	**THE SIXTH STREET VIADUCT, KANSAS CITY.**	
	By **E. E. Howard**	42
	Discussion:	
	By Daniel Bontecou	95
	Victor H. Cochrane	96
	O. E. Mogensen	98
	N. T. Blackburn	100
	George H. Pegram	101
	E. E. Howard	101
1121	**SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.**	
	By **W. S. McFetridge**	105
	Discussion:	
	By Clinton S. Bissell	131
	F. Lavis	133
	E. W. Lewis	142
	George L. Dillman	143
	W. S. McFetridge	144
1122	**COMPUTATION OF STRESSES IN OPEN-WEBBED ARCHES WITHOUT HINGES.**	
	By **C. W. Hudson**	145
1123	**HYDRO-ELECTRIC POWER IN CANADA.**	
	By **Cecil B. Smith**	154
	Discussion:	
	By H. Holgate	194
	E. J. Beugler	199
	Cecil B. Smith	200

No. PAGE

1124 **TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.**

By **Arthur N. Talbot and Herbert F. Moore** 202

Discussion:

By JOHN C. MOSES 246
ARTHUR N. TALBOT and HERBERT F. MOORE 247

1125 **IMPURITIES IN SAND FOR CONCRETE: AN INFORMAL DISCUSSION.**

By SANFORD E. THOMPSON 250
R. W. LESLEY 258
C. P. HOWARD 260
RICHARD L. HUMPHREY 260
G. S. DAVISON 262
S. WHINERY 262
CHARLES M. MILLS 263
T. F. RICHARDSON 265
E. G. HAINES 267
SANFORD E. THOMPSON 271

1126 **FIRE-RESISTANT CONSTRUCTION OF BUILDINGS: AN INFORMAL DISCUSSION.**

By HERBERT M. WILSON 274
R. W. LESLEY 290
RICHARD L. HUMPHREY 291
S. WHINERY 293

1127 **THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.**

By **C. E. Grunsky** 294

Discussion:

By W. C. HAMMATT 385
ROBERT G. DIECK 385
CHARLES E. GREGORY 390
E. KUICHLING 395
KENNETH ALLEN 400
WALTER N. FRICKSTAD 405
C. E. GRUNSKY 412

1128 **WALNUT LANE BRIDGE, PHILADELPHIA.**

By **George S. Webster and Henry H. Quimby** 423

1129 **HAS EXPERIENCE DEMONSTRATED THAT THE OILING OF ROADS IS THE MOST SATISFACTORY OR ECONOMICAL METHOD OF PREVENTING DUST AND PRESERVING THE ROAD SURFACE? AN INFORMAL DISCUSSION.**

By S. WHINERY 462
G. S. DAVISON 466

No.		PAGE
1130	**CONCRETE PILES.**	
	By **Howard J. Cole**..	467
	Discussion:	
	By MORTON L. TOWER	488
	L. J. MENSCH	489
	CHARLES H. HIGGINS	491
	MAXWELL M. UPSON	494
	THOMAS C. DESMOND	498
	R. D. COOMBS	502
	HOMER A. REID	503
	HUNLEY ABBOTT	507
	HOWARD J. COLE	509

MEMOIRS OF DECEASED MEMBERS

GEORGE ADGATE, M. AM. SOC. C. E.	514
WILLIAM PRICE CRAIGHILL, PAST-PRESIDENT, AM. SOC. C. E.	517
CHARLES HERMANY, PAST-PRESIDENT, AM. SOC. C. E.	525
CABELL BRECKINRIDGE, M. AM. SOC. C. E	529
WALTER LORTON DUSENBERRY, JUN. AM. SOC. C. E.	533

PLATES

PLATE		PAPER	PAGE
I.	Portions of Profile and Plan, Sixth Street Viaduct, Kansas City	1120	45
II.	Views of Sixth Street Viaduct and Kaw River Bridge	1120	53
III.	Concrete Piles, and Pile Drivers	1120	59
IV.	Pedestal, and Tower Span, etc., Sixth Street Viaduct	1120	61
V.	Detail of Pier 2, Kaw River Bridge	1120	67
VI.	Details of Columns and Bracing, Sixth Street Viaduct	1120	71
VII.	Gallows Frame for Erection of Long Girders; and Concrete Floor, Sixth Street Viaduct	1120	77
VIII.	Concrete Plant for Laying Floor of Sixth Street Viaduct; and Finished Floor of Approaches	1120	81
IX.	Map Showing Lines of Little Kanawha R. R., in West Virginia	1121	109
X.	Map of the Dominion of Canada	1123	157
XI.	Map of Part of the Province of Quebec	1123	199
XII.	Application of Instruments for Measuring Stresses, etc., in Compression Pieces; and Test Train on White Heath Bridge	1124	209
XIII.	Plan of Part of San Francisco, Cal., Showing North Point Main Relief Outlets	1127	371
XIV.	Walnut Lane Bridge, Philadelphia	1128	427
XV.	General Plan and Elevation of Walnut Lane Bridge and Approaches	1128	429
XVI.	Falsework and Main Arch Centering, Walnut Lane Bridge	1128	431
XVII.	Plan and Elevation, and Arch Details, Walnut Lane Bridge	1128	433
XVIII.	Construction of Arches, Walnut Lane Bridge	1128	435
XIX.	Cross-Sections, Walnut Lane Bridge	1128	437
XX.	Moving Centering, and View of Voussoirs, Walnut Lane Bridge	1128	441
XXI.	East Abutment, Walnut Lane Bridge	1128	445
XXII.	Floor Details, Walnut Lane Bridge	1128	447
XXIII.	Brackets and Piers, Walnut Lane Bridge	1128	449

PLATE		PAPER	PAGE
XXIV.	End of Centering, and View Between Ribs, Walnut Lane Bridge	1128	451
XXV.	Floor System and Deck View, Walnut Lane Bridge	1128	453
XXVI.	Views of Walnut Lane Bridge	1128	455
XXVII.	Centering, Walnut Lane Bridge	1128	457
XXVIII.	Stress Diagram, Walnut Lane Bridge	1128	459
XXIX.	Approach Spans, Walnut Lane Bridge	1128	461
XXX.	Timber Piles Damaged in Driving, and Reinforced Concrete Piles, Type A, Made-in-Place Class	1130	469
XXXI.	Reinforced Concrete Piles, Type A, Made-in-Place Class, and Moulded Piles	1130	473
XXXII.	Vertically-Moulded Piles of Reinforced Concrete for Boardwalk, Atlantic City, N. J.	1130	477
XXXIII.	Reinforced Concrete Piles, Type B, Made-in-Place Class	1130	479
XXXIV.	Reinforced Concrete Piles, Type B, Made-in-Place Class, and Type 3, Moulded Class	1130	481
XXXV.	Reinforced Concrete Piles, Type B, Made-in-Place Class, and Type 3, Moulded Class	1130	483
XXXVI.	Reinforced Concrete Piles, Type 3, Moulded Class	1130	485
XXXVII.	Views of Concrete Pile with Enlarged Base	1130	507
XXXVIII.	Method of Driving, and Pulling Out Concrete Pile Casing	1130	509

Portrait of the late William Price Craighill, Past-President, Am. Soc. C. E. . . 516

Portrait of the late Charles Hermany, Past-President, Am. Soc. C. E. 524

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1118

CAISSON DISEASE AND ITS PREVENTION.*

By Henry Japp, M. Am. Soc. C. E.

With Discussion by Messrs. J. S. Haldane, Frederick L. Keays, Albert J. Loomis, William L. Saunders, Walton I. Aims, Seward Erdman, T. Kennard Thomson, and Henry Japp.

The prevention of caisson disease has not received as much attention as its cure, the general opinion being that it cannot be prevented. In forming an idea as to the possibility of its prevention, something may be learned from Nature's method of carrying out work by the aid of living organisms under air pressure.

First let us inquire whether Nature has a sufficient range of air pressure to make it necessary for a capacity to sustain variations without injury. The highest altitude on this earth is Mount Everest, 29 000 ft. above the level of the sea, and the deepest mine is Tamarack No. 3 Shaft, of the Calumet and Hecla Mines, 4 407 ft. below sea level.

The curve on Fig. 1 shows the absolute air pressures at various altitudes between these extremes. The total difference of 12½ lb. per sq. in., or from 9¾ lb. below atmospheric pressure at sea level (viz., 14.7 lb.) up to 2¾ lb. above normal, is considerable.

Mount Everest has never been ascended, but an elevation of 20 000 ft. has been reached, and men can live in comfort at an altitude of 7 000 ft., as in Mexico City, which has an air pressure of 3½ lb.

* Presented at the meeting of June 23, 1909.

per sq. in. less than at the coast. Such a change of pressure is quite noticeable to those whose Eustachian tubes are closed, and, although the range of pressure is insufficient to produce caisson disease, yet travelers have complained of discomfort, not unlike a very mild attack, after a rapid journey by rail from Vera Cruz to Mexico City.

Eagles are known to go to great heights beyond the range of human vision, fish are capable of going to great depths in water, and beasts of prey roam from the mountain side to the valleys far below. With each descent these creatures have undergone compression, and with each ascent decompression.

It will be noted that Nature has no time limit for immersion under increased pressure, and the application of pressure is a much speedier and easier operation than that of decompression; and, while the compression is down-hill and attained with little effort, the decompression can only be attained by the continuous effort of climbing, and, in long ascents, with frequent rests or by stages. This is surely a forecast of the stage-decompression theory.

From Nature then, our best teacher, we learn that the time taken to enter the air-chamber may be short, the time spent in the air-chamber may be long, and the rate of coming out must be slow and accompanied by exercise.

In early compressed-air work engineers were confronted with caisson disease, and have long been striving for a remedy. Men of observation noted first that lock tenders who made frequent entry to the air-lock for short visits were much more immune from the malady than workers who stayed for a full shift, and from this argued that shortening the time in the air-chamber would be a safeguard, and, as higher pressures were used, the hours continued to be shortened until it was thought that half an hour at a maximum of 50 lb. gauge pressure was the limit of human endurance.

As the number of compressed-air works increased, the workmen gained in experience, and it was noted that workmen of experience who suffered from the painful forms of caisson disease relieved the pain by re-entering the air-chamber.

About thirty years ago Paul Bert advanced the theory that caisson disease was caused by the nitrogen of the air being dissolved in the blood. He advocated slow decompression, suggesting 30 min. for 2 to 3 atmospheres (15 to 30 lb. gauge pressure) and 60 min. for 3 to 4

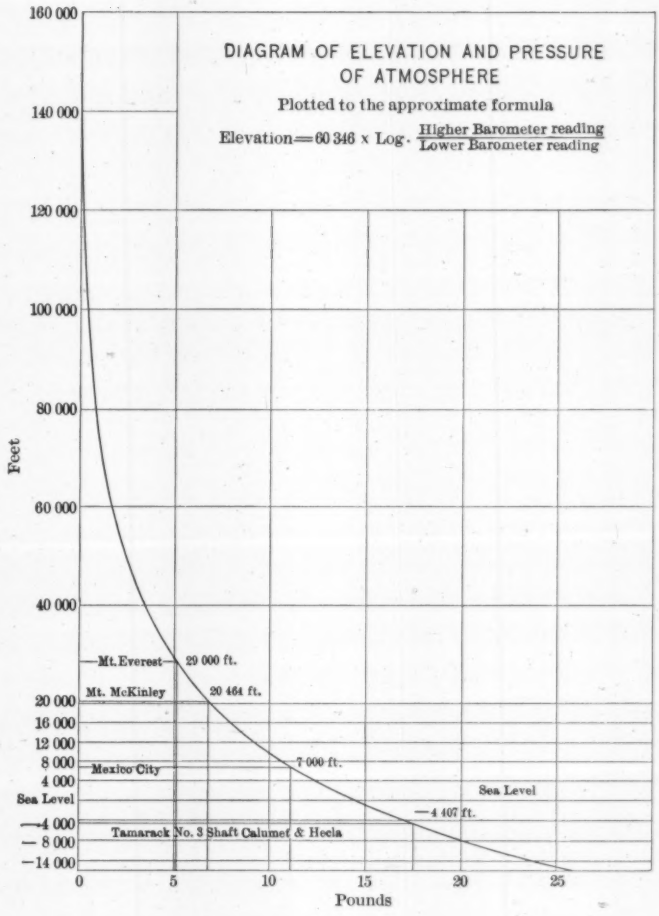
DIAGRAM OF ELEVATION AND PRESSURE OF ATMOSPHERE
Plotted to the approximate formula
Elevation = 60 346 x Log. Higher Barometer reading / Lower Barometer reading
Feet
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
16 000
12 000
8 000
4 000
Sea Level
—4 000
— 8 000
—14 000
Mt. Everest
29 000 ft.
Mt. McKinley
20 464 ft.
Mexico City
7 000 ft.
Sea Level
—4 407 ft.
Tamarack No. 3 Shaft Calumet & Hecla
0
5
10
15
20
25
Pounds

FIG. 1.

atmospheres (30 to 45 lb. gauge pressure). Since his time many writers and experimenters have followed in his footsteps, but it was not until Dr. Smith, of New York, suggested that a recompression chamber would act as a remedy, and E. W. Moir, M. Am. Soc. C. E., of London, without knowledge of Dr. Smith's prior suggestion, actually built a medical air-lock, or recompression chamber, in 1890, on the old Hudson Tunnel, New York, that any reliable cure for the trouble was discovered. Fig. 2 shows the medical air-lock designed by Mr. Moir, of which six were used on the East River tunnels of the Pennsylvania Tunnel and Terminal Railroad.

It was then found that if a man suffering from caisson disease in any of its many forms, varying from unconsciousness and paralysis to an acute pain in the limbs, was quickly recompressed and allowed to decompress slowly, in many cases a cure resulted, but not always.

This went to prove that the disease was a mechanical one, and was caused by the dissolved air in the blood and tissues becoming liberated in the body in the form of expanding bubbles which tore the tissues, caused pressure on the brain, injured the spinal cord, or frothed up the blood and stopped the circulation and heart action.

Much has been done since the introduction of the medical air-lock. The percentage of carbon dioxide in the air of the working chamber has been kept within safe limits, moisture and oil have been extracted from the air before delivery to the chamber, and the workmen have been well cared for; but still caisson disease claims its victims, and engineers consider this inevitable.

The erratic manner of the disease puzzles doctors and engineers alike. Men who have worked for months in high pressures for the regulation shift without suffering have been suddenly attacked and died; others, working only half a shift, have been paralyzed; while exceptions have worked for 12 hours at a time and have suffered only slight pains which have quickly passed away.

The writer, as Managing Engineer for S. Pearson and Son, Inc., on the East River Tunnels contract for the Pennsylvania Tunnel and Terminal Railroad Company, New York, with as many as ten tunnel headings under compressed air simultaneously, has had many opportunities of studying this strange disease.

When these tunnels were commenced, in 1904, the knowledge of compressed air at the disposal of the contractor was quite extensive,

MEDICAL AIR LOCK USED IN THE EAST RIVER TUNNELS OF THE PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD

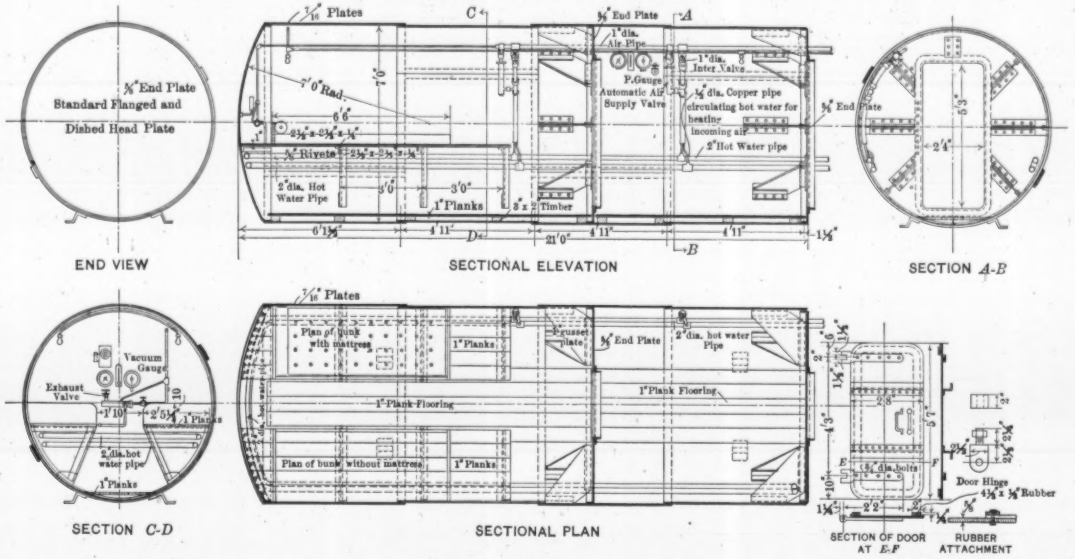

FIG. 2.

and included the long experience of Alfred Noble, Past-President, Am. Soc. C. E., on caisson work in the United States, and Mr. E. W. Moir's experience on the Forth Bridge caissons, the old Hudson Tunnel, and the Blackwall caissons and tunnel.

In the light of past experience, the general conduct of the work was framed on a few established rules which when condensed amount to the following: No workman was allowed to enter the air-chamber without a physical examination by the qualified medical officer of the contractors. Sound physique was the chief requirement. The men were cautioned not to enter the air on an empty stomach, to wear warm clothing on coming out, and to drink hot coffee.

The time worked in the air-chamber was limited to 8 hours with half an hour off for lunch, up to 32 lb. gauge pressure, and two spells of 3 hours each with 3 hours rest between for pressures from 32 to 42 lb., and two spells of 2 hours each for pressures greater than 42 lb. with 4 hours rest between, with no limitation as to decompression. Two medical air-locks were installed on each side of the river, well-warmed dressing rooms were provided for the workmen, and there were covered gangways for access to the shafts.

The air was cooled before delivery to the tunnels, and samples were taken in the tunnels by Mr. Noble's engineers and analyzed daily. The air was regulated so that the carbon dioxide did not exceed 10 parts in 10 000, and the tunnels were kept in a sanitary condition.

Owing to the grade of the tunnels being so deep on the Manhattan side of the river, the air pressure very quickly rose to 36 lb.

Practically no cases of bends occurred until the pressure reached 29 lb., and then, within a few days of each other, two men died. These men had entered the air-chamber without being passed by the doctor. Then it became necessary to post outside each air-chamber a guard whose duty it was to keep out of the tunnel men who had no doctor's pass. At this time, reliance could not be placed on the tunnel foremen, as they were likely to be absent from work, and new men had to be selected each day.

For many months after the work started, while the men were being seasoned, the tunnel gangs and foremen were in a state of change, owing to the difficulty of getting good men and the frequent absences due to caisson disease, and it was a long time before an efficient organization was built up.

As the tunnels were driven deeper beneath the East River the pressure quickly rose, and ultimately reached 36 lb., gauge pressure, with only one set of air-locks in operation; but even the change at 32 lb. from one shift of 8 hours to two shifts of 3 hours each gave no relief, and cases of bends, sometimes fatal, continued all the time.

It was not long after 27 lb. was reached that the more sensitive members of the staff found that it did not pay to come out quickly, and at 30 lb. pressure it became a custom to take about ½ min. for each pound. After one or two additional fatal cases occurred, it was decided to limit the workmen to approximately the same rate of decompression, or actually 15 min. for 35 lb. pressure.

Many of the men complained that taking so long to decompress gave them caisson disease, and it was difficult to compel them to take long enough. The guards at the entrance to the tunnels had now to record the time taken to decompress, and, as the workmen frequently used the lower muck locks as well as the upper man lock, it was impossible to tell when decompression commenced owing to the noise of exhausting air. Therefore it became necessary to run a small ¼-in. pipe from the exhaust pipe of each lock to the cabin in which the guard was stationed, a whistle was attached, and a small ball of cotton was suspended by a light string over each pipe. The clerk, noting when the ball was puffed off for each lock, booked the workmen off as they left the lock. The material locks were fitted with inner material valves and, in addition, man valves of smaller size, and a pressure gauge and a clock were fixed in each air-lock.

As the guard had already booked the men as they entered, he could tell if any were exceeding the regular working shift, and his record was valuable for checking the time-keepers. This rough method of checking the duration of the decompression was quite good enough for the purpose. An attempt to improve it was made when a 12-in. Crosby recording gauge was installed on one air-lock, but it was ineffectual because the air-lock was often sent out or decompressed with no one inside, and this complicated the record, which was much too small, and involved considerable trouble in locating the record of decompression for individual gangs. No doubt a suitable recording instrument could be devised for this special purpose.

The effect of lengthening the decompression period to 15 min. reduced the number of cases of bends, and no doubt prevented many

fatal ones, but they still occurred. As the tunnels had many months to run at high air pressures, the question was: What else can be done to prevent them?

The writer, on coming out of the tunnel with the workmen, observed that the rate of decompressing was most irregular. One lock tender would allow the air to escape slowly until the 15 min. was almost exhausted, and then, by opening the valve, would let off the remaining pressure very quickly. Others would reverse the process, and exhaust quickly, and then keep the men under 2 or 3 lb. until the time expired. To avoid this, a simple decompressing valve was designed (Fig. 3) which gave a uniform decompression from 35 lb. to atmosphere in 15 min. A somewhat similar one (Fig. 4) was designed for the medical air-lock, for 1 hour decompression for 35 lb., with an automatic ventilator attached.

These valves certainly improved conditions, but still fatal cases occurred. After the first valve was under operation, the writer's attention was called to "Modern Tunnel Practice," by D. McN. Stauffer, M. Am. Soc. C. E., wherein a description is given of a needle decompression valve used on the air-locks at the Kiel Dry Dock Works, in Germany. A similar valve was also used for compression. On entering the lock the air was admitted at the rate of 1.5 lb. per min., and was decompressed at the rate of ¾ lb. per min., or, for 35 lb. gauge pressure, 23 min. for entering the air and 46 min. for leaving. This needle-valve was often frozen up, but otherwise worked well.

Such speeds seemed altogether too slow, and Mr. Stauffer in his book states that "these rates would be deemed excessively slow in American caisson practice." No date is given for the Kiel Dry Dock work, but presumably it was under way in 1904.

It was not thought advisable to increase the time of decompression at that time, but preliminary tests under air pressure in the tunnels for 1½ hours were then tried for green men, followed by a second medical examination after decompressing in 15 min. A few men were eliminated by this test, and one case of permanent paralysis resulted from the test in 34 lb. gauge pressure. Fresh starters were made to stay in the tunnel for one-quarter of a shift for the first half, and if no caisson disease followed they were allowed to work for the second half. This proved a good safeguard.

AUTOMATIC CONSTANT-RATE DECOMPRESSION VALVE

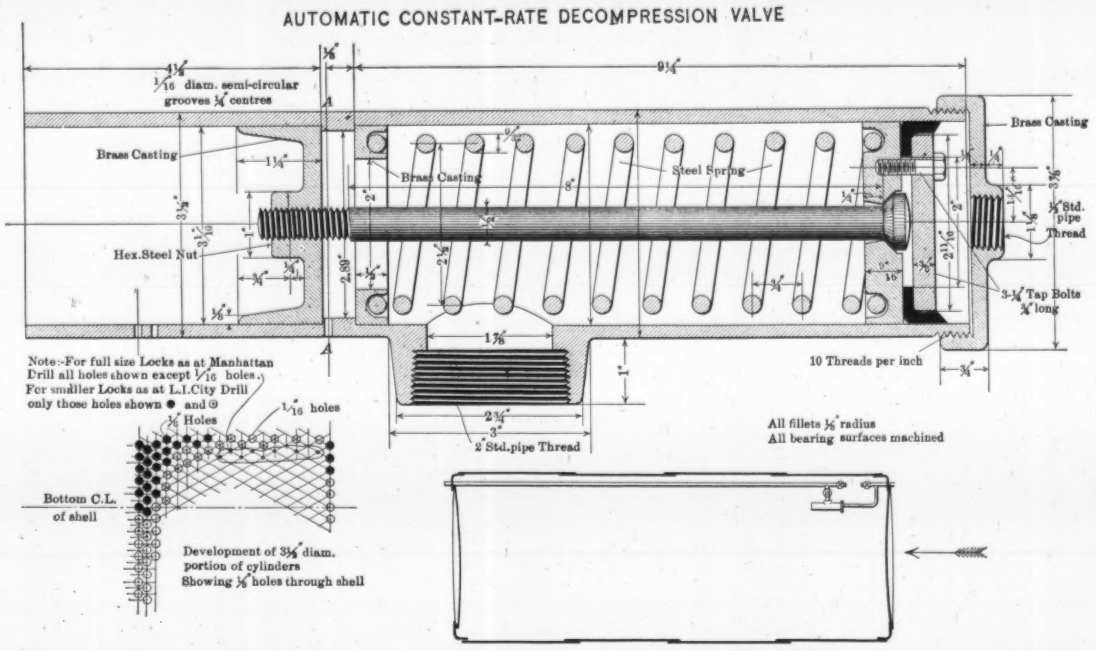

DIAGRAM OF CONNECTIONS

FIG. 3.

On November 8th, 1906, a second bulkhead was put in operation in one of the tunnels, and the pressure between the two bulkheads was reduced to 15 lb. gauge. The number of cases of disease was very small for that tunnel, and as soon as possible additional bulkheads were placed in all four tunnels. The result was to have been expected from the experience in other tunnels where two bulkheads were used; the exercise of the long walk between bulkheads, at low pressure, seemed to assist in driving off the bubbles of air from the blood.

The workmen were allowed to decompress from 35 to 15 lb. as quickly as they pleased. They then walked for 500 ft. along the tunnel under 15 lb., taking at least 5 min., and then decompressed from 15 lb. to atmosphere in 10 min., so that in all about 16 min. were occupied in decompressing. When the inner pressure was less than 32 lb. an 8-hour shift was worked, with ½-hour interval for lunch, between bulkheads in low air pressure.

Just when it looked as if the double bulkheads with stage decompression had eliminated fatal and severe cases of caisson disease, two deaths occurred in physically perfect subjects.

In order to discover, if possible, the connection between the cases of disease and other things, charts were plotted by the medical staff for some months, showing the rise and fall of air pressure, hours worked, humidity, temperature, percentage of carbon dioxide, and number of green men in the tunnel, along with barometer readings, condition of weather, direction of wind, and number of cases of bends. The results were not very encouraging, but it was noted that the number of green men, the height of pressure in the tunnel and the number of cases of bends varied together.

The percentage of cases in air pressure of 31½ lb. for 8-hour shifts was no more than the percentage in 32½ lb. for two 3-hour shifts—in fact, it was, if anything, less for the longer shift. The decrease in length of shift added one extra gang of men, and probably many of these men being green accounted for this.

In November, 1906, Mr. Moir became acquainted in part with Dr. Haldane's work of investigation for the British Admiralty on "Deep Sea Diving," and ordered the decompressing valves changed so that the decompression was accelerated at the commencement of the operation, and slowed off toward the end. In October, 1906, the writer, endeavoring to find out the effect of sudden decompression,

AUTOMATIC CONSTANT RATE DECOMPRESSION AND VENTILATING VALVE

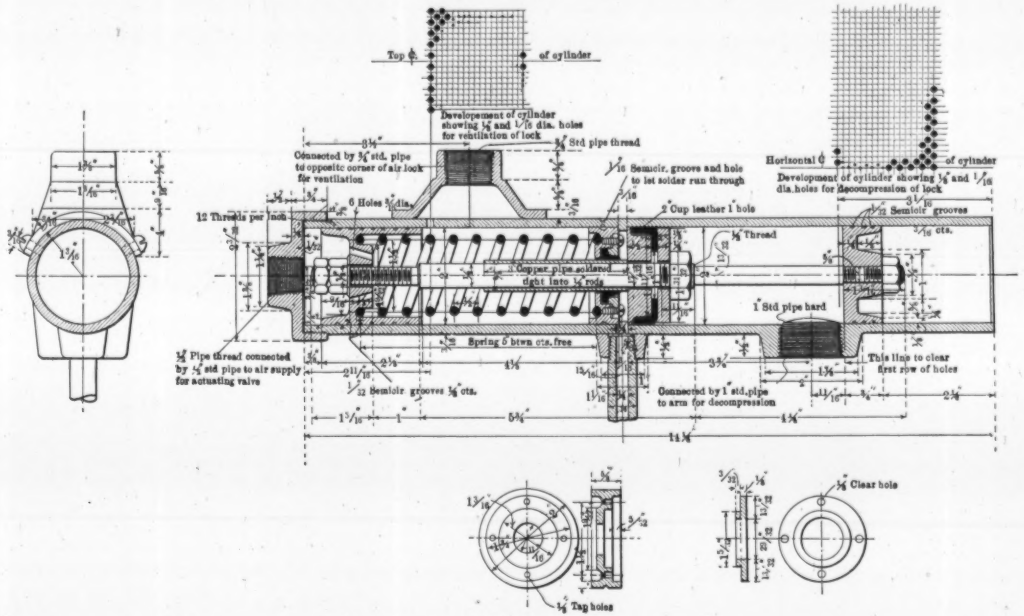

FIG. 4.

fitted an ordinary glass siphon bottle with a valve and pressure gauge. This bottle was partly filled with water, and put under 36 lb. air pressure for 12 hours. On suddenly releasing the pressure, no effervescence whatever took place, and only a very few minute bubbles were visible. The same thing was tried with the bottle partly filled with bullock's blood, with like results. Arguing that the placid surface of blood in a stationary vessel offered no such opportunity for dissolving air as occurs in the lung surface with the blood circulating, the siphon was again charged with 36 lb. pressure, but was rotated for 24 hours under this pressure. The same result as before was observed on suddenly releasing the pressure; but, on closing the valve, the pressure recorded on the gauge was about 1 lb. per sq. in. in less than 1 hour, and in 6 days a pressure of 13 lb. was recorded, showing that the air dissolved in the blood came off very slowly.

If the cork of a bottle of aerated water is drawn quickly, it effervesces so violently that the water froths out of the neck of the bottle; but, if the gas is allowed to escape slowly, only a mild bubbling results. This in general is the theory on which slow decompression has been advocated in the past. The violence of the escape of carbon dioxide gas from aerated water, as compared with the escape of air from water released from pressure, is due to the greater solubility of carbon dioxide than air.

It was difficult to know what else could be done at this time in the way of eliminating caisson disease on the East River tunnels, and the cases of disease were certainly less frequent and less severe, although at long intervals fatal cases still occurred. The workmen, by this time, had become seasoned, and more seasoned men were now available from other works.

Dr. Leonard, Hill read a paper on deep sea diving and caisson work, on September 26th, 1907, and exhibited the web of a living frog under a pressure of 20 atmospheres. On suddenly decompressing, bubbles were plainly visible in the blood vessels, multiplying quickly as they traveled along until the circulation was choked; on recompression the bubbles disappeared and circulation was resumed. He stated that he and Mr. Greenwood had been under an air pressure of 92 lb., and had suffered no ill effects.

Dr. J. S. Haldane, on November 29th, 1907, read a most enlightening paper before the Society of Arts, in London, in which he gave

the results of his investigations for the British Admiralty on "Deep Sea Diving," in which he was assisted by Dr. Boycott and Lieutenant Damant.

He set forth very clearly the question of the solution of gases by the blood, fat, and tissues of the body, and attempted to determine the time taken to saturate the blood with the nitrogen of the air. He proved conclusively that the quantity of carbon dioxide adjusts itself to a constant percentage inversely proportional to the pressure in the alveoli, or air cells.

In atmospheric pressure, the percentage of carbon dioxide in the alveolar or expired air is 5.6%, and at a pressure of 2 atmospheres, absolute, this is reduced to 2.8%; so that the question of the percentage of carbon dioxide in the air of the working chamber is not important unless it approaches the percentage of carbon dioxide in the air cells of the lungs.

For instance, if the air-chamber is under an air pressure of 30 lb., or 3 atmospheres absolute, the percentage of carbon dioxide in the air cells is 5.6 divided by 3, or 1.86%; and, if the percentage of carbon dioxide in the air-chamber does not exceed 1%, no ill effects will arise. This is ten times as much as is generally specified, namely, 10 parts in 10 000, and greatly reduces the amount of compressed air necessary per man per hour, which can be calculated approximately from the following simple formula:

$$\text{Cubic feet per man per hour} = \frac{80 \text{ cu. ft.}}{\text{Percentage } CO_2 \text{ permitted}}.$$

Thus, if 0.04% is the CO_2 in the atmosphere, and the percentage in the tunnel is allowed to go up to 0.10%, the air required per man per hour $= \frac{80}{0.06} = 1\,333$ cu. ft.

Dr. Haldane has completely removed all difficulty of respiration and discomfort of divers at great depths through the application of this theory regarding the percentage of CO_2 by supplying the diver with the same volume of compressed air per minute at different levels instead of the same volume of free air per minute.

The question of ventilating compressed-air tunnels seldom arises, as the volume of air generally far exceeds the amount required for purity, on account of the tremendous leakage at the face, so that this is not nearly as important as the solubility of air by the blood and tissues of the body.

After demonstrating that the CO_2 is dealt with automatically, he shows that nearly all the oxygen dissolved by the blood is taken up chemically by the hemoglobin, but the nitrogen remaining is incapable of combination with the system, and is dissolved by the blood and tissues. These, therefore, become saturated with nitrogen during immersion in compressed air.

If saturation takes place quickly it will not matter how long a workman stays under air pressure; but, if slowly, then shortening of the working shift will be a safeguard.

The rate of saturation is the same as the rate of desaturation if compression and decompression are instantaneous; so that, by altering the time for decompression, it is possible to determine by experiment the rate of saturation and desaturation.

Dr. Haldane experimented with men and goats at high air pressures, with varying lengths of stay under air pressure, and with different speeds of decompression. From these he concluded that certain parts of the body, where the circulation is rapid and the number of blood vessels high for the mass of the part supplied, would be half saturated or desaturated in 5 min., while other parts, with slight circulation for the mass, would require 75 min. for 50% saturation, especially the fatty parts, as fat is found to dissolve about six times as much nitrogen as blood.

Fig. 5 shows the rate of saturation, on the basis of half saturation in various times from 5 to 75 min., and curves of desaturation, if the decompression is instantaneous after varying times of immersion from ½ hour up to 8 hours.

It will be seen that 90% saturation takes place in the slowest parts of the body in about 4 hours, and complete saturation for the quickest parts in about 40 min., so that after 40 min. immersion there is danger in too rapid decompression.

Dr. Haldane has noted that no serious cases of caisson disease have occurred for rapid decompression at a working pressure of 19 lb. gauge pressure, or 2.3 atmospheres, absolute. Therefore, he concludes that it is always safe to decompress rapidly to half the absolute pressure, or even to the absolute pressure divided by 2.3, without bubbles being liberated. Further, if the decompression for the remaining pressure is continued at such a rate as will keep pace with the rate of desaturation, then, the absolute pressure of nitrogen in the blood never exceed-

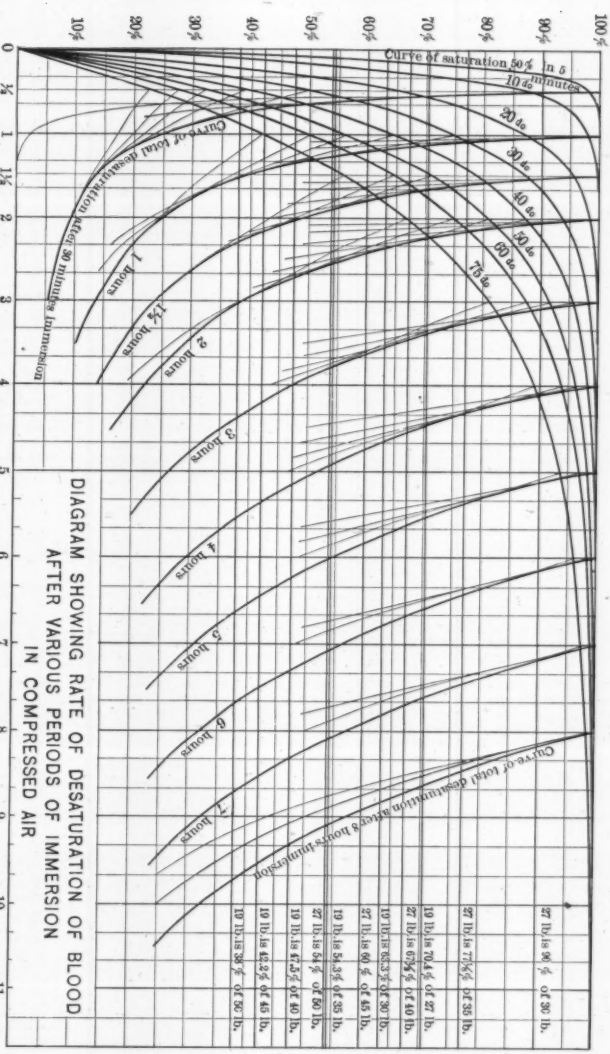
DIAGRAM SHOWING RATE OF DESATURATION OF BLOOD
AFTER VARIOUS PERIODS OF IMMERSION
IN COMPRESSED AIR
Hours
Curve of saturation 50 lb. in 5 minutes
10 do.
20 do.
30 do.
40 do.
50 do.
60 do.
75 do.
1 hours
1¼ hours
2 hours
3 hours
4 hours
5 hours
6 hours
7 hours
Curve of total desaturation after 30 minutes immersion
Curve of total desaturation after 6 hours immersion
27 lb. is 96 % of 30 lb.
27 lb. is 75½% of 35 lb.
19 lb. is 70.4 % of 27 lb.
27 lb. is 67½% of 40 lb.
19 lb. is 63.3 % of 30 lb.
27 lb. is 60 % of 45 lb.
19 lb. is 54.3 % of 35 lb.
27 lb. is 54 % of 50 lb.
19 lb. is 47.5 % of 40 lb.
19 lb. is 42.2 % of 45 lb.
19 lb. is 38 % of 50 lb.

Fig. 5.

ing 2.3 times the absolute pressure in the air-lock during decompression or after returning to the atmosphere, no symptoms will occur.

Table 1 shows the rates of decompression advocated by Dr. Haldane for caisson workers:

TABLE 1.—DR. HALDANE'S RATE OF DECOMPRESSION IN CAISSON AND TUNNEL WORKS.

Working pressure, in pounds per square inch.	NUMBER OF MINUTES FOR EACH POUND OF DECOMPRESSION AFTER THE FIRST RAPID STAGE.		
	After first 3 hours' exposure.	After second or third 3 hours' exposure showing an interval for a meal.	After 6 hours or more of continuous exposure.
18-20	2	3	5
21-24	3	5	7
25-29	5	7	8
30-34	6	7	9
35-39	7	8	9
40-45	7	8	9

If the pressure is 40 lb., gauge, decompress rapidly from 40 lb. to $\frac{40 + 15}{2} = 27\frac{1}{2}$ lb., absolute, or $12\frac{1}{2}$ lb., gauge, in 3 min., and then take 7 min. for each remaining pound, viz., $12\frac{1}{2}$ lb., or $87\frac{1}{2}$ min. plus 3 min. = $90\frac{1}{2}$ min., or $1\frac{1}{2}$ hours in all for a 3-hour immersion; or, for a 6-hour immersion, 115 min. in all.

The upper diagram on Fig. 6 shows this graphically.

It will be seen that, if this rule is followed, the condition of the absolute pressure, being half the pressure in the blood, is obtained. If uniform decompression for the same time were adopted, the decompression would be too slow for the earlier part, would very much retard the desaturation, and would give a greater tension in the blood than 19 lb., on coming out of the air pressure.

It is obvious that while the air-lock has any pressure in it the desaturation takes place only in relation to that pressure, and, if one adopts Dr. Haldane's idea of keeping a portion of the tunnel at half the absolute pressure for a dressing room to act as a purgatorial chamber, where the men may wash and change while desaturating down to 19 lb., we must make a new curve for desaturation and increase the time of decompressing, as will be seen by the second diagram on Fig. 6.

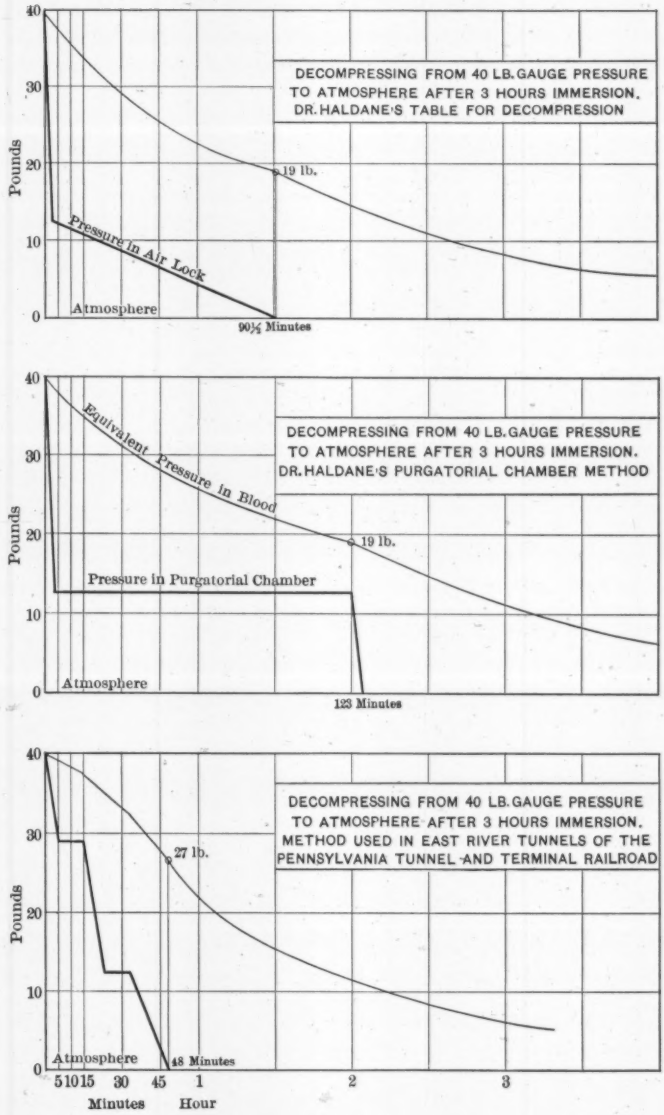
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. DR. HALDANE'S TABLE FOR DECOMPRESSION
Pounds
40
30
20
10
0
19 lb.
Pressure in Air Lock
Atmosphere
90½ Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. DR. HALDANE'S PURGATORIAL CHAMBER METHOD
Equivalent Pressure in Blood
19 lb.
Pressure in Purgatorial Chamber
Atmosphere
123 Minutes
DECOMPRESSING FROM 40 LB. GAUGE PRESSURE TO ATMOSPHERE AFTER 3 HOURS IMMERSION. METHOD USED IN EAST RIVER TUNNELS OF THE PENNSYLVANIA TUNNEL AND TERMINAL RAILROAD
27 lb.
Atmosphere
48 Minutes
5 10 15 30 45 1 2 3
Minutes Hour

FIG. 6.

From this it appears that the tension in the blood does not reach 19 lb. until a period of 123 min. has been passed in the purgatorial chamber after 3 hours' immersion.

It will be noticed that Dr. Haldane bases his theory of decompression on the fact that no cases of caisson disease are chronicled for men working in gauge pressures up to 19 lb., or 34 lb., absolute, and if it is safe to decompress suddenly from 34 lb., absolute, to 15 lb., absolute, it is safe to decompress suddenly from 50 lb., absolute, to 22 lb., absolute, or from 35 lb., gauge pressure, to 7 lb., gauge pressure.

The question arises at once: Are the times suggested by Dr. Haldane practical?

Many caissons of small dimensions are sunk under air pressures of 35 lb. and to cramp men in a small air-lock for 73 min. is out of the question, and the fitting of large compartments in small vertical air-locks, as used in building foundations in New York City, would be difficult. In tunnel work it would be easier, but the present method of rapid decompression is so radically different from Dr. Haldane's suggestion that it quite appals one to think of taking so long.

The experience of most compressed-air works is that up to 27 lb. gauge pressure there is very little trouble, and, using this as the safe limit, the times required to reduce the saturation in the blood to 27 lb. is not so excessive for pressures up to 50 lb.

The horizontal lines on Fig. 5 show the equivalent points for 27 and 19 lb. in the blood for 50, 45, 40, 35, 30, and 27 lb., gauge pressure.

It will be noticed that the slowly saturated parts of the body are only partly saturated in the first few hours of immersion, and therefore the desaturation is proportionately fast.

From Fig. 5, Tables 2 and 3 have been compiled; they show the risks taken in rapid decompression after long immersion.

If the decompression is slow, these curves will be flattened out, and a longer time will elapse before the saturation of the blood falls to the equivalent of 19 lb. or 27 lb., as the case may be.

Fig. 7 shows the curves of decompression from 27, 30, and 32 lb., gauge, after 8 hours' immersion, and for 32, 35, 40, and 42 lb., gauge, after 3 hours' immersion, and 42, 45, and 50 lb., gauge, after 2 hours' immersion, on the basis that it is safe to decompress from 27 lb., gauge pressure, in 9 min. This reduces the saturation in the blood to the equivalent of 25 lb. on reaching atmospheric pressure.

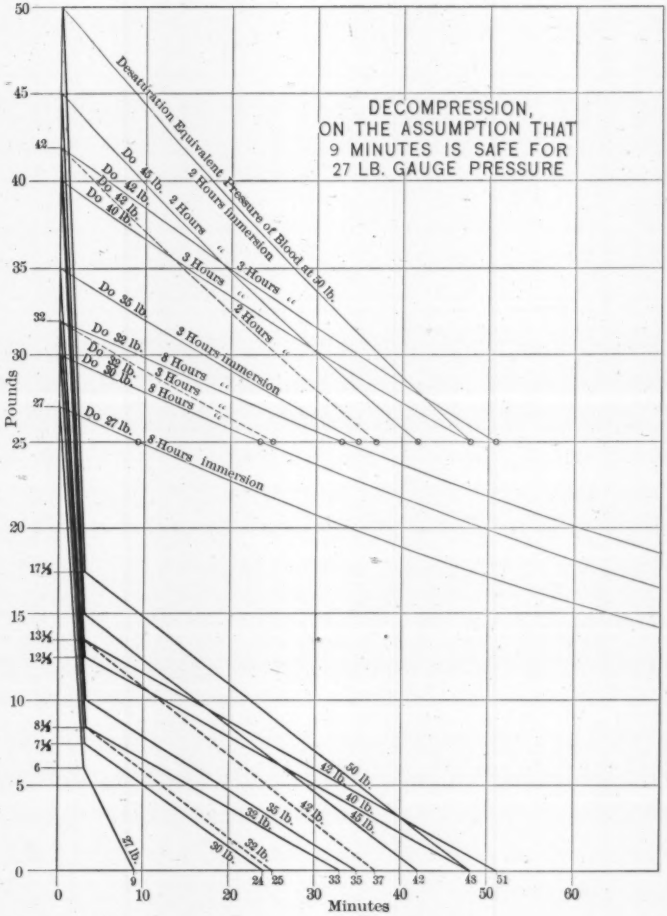
DECOMPRESSION,
ON THE ASSUMPTION THAT
9 MINUTES IS SAFE FOR
27 LB. GAUGE PRESSURE
Desatiration Equivalent Pressure of Blood at 50 lb.
Do 45 lb. 2 Hours immersion
Do 42 lb. 2 Hours "
Do 40 lb. 2 Hours "
Do 35 lb. 3 Hours "
Do 32 lb. 3 Hours "
Do 30 lb. 3 Hours "
Do 27 lb. 3 Hours "
3 Hours immersion
3 Hours "
3 Hours "
8 Hours immersion
8 Hours "
8 Hours "
8 Hours "
Pounds
Minutes
50 lb.
42 lb.
40 lb.
45 lb.
35 lb.
42 lb.
32 lb.
30 lb.
27 lb.

FIG. 7.

TABLE 2.—Times (in Minutes) Required for Pressure in Blood to Fall to the Equivalent of 19 Pounds Gauge Pressure with Instant Decompression after Various Periods of Immersion in Compressed Air.

No. of hours of immersion.	Gauge Pressures:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour	4	5	8	10	14	16½
1 "	7	10	15	20	26	32
1½ hours	10	16	24	32	40	48
2 "	14	21	32	42	52	62
3 "	22	30	45	59	72	82
4 "	28	39	56	69	82	93
5 "	33	44	60	74	86	98
6 "	35	47	63	76	89	100
7 "	37	48	64	78	90	102
8 "	38	50	66	79	92	103

The times in Table 2, being for instant decompression, must be suitably extended for slow decompression.

TABLE 3.—Times (in Minutes) Required for Pressure in Blood to Fall to the Equivalent of 27 Pounds Gauge Pressure with Instant Decompression after Various Periods of Immersion in Compressed Air.

No. of hours of immersion.	Gauge Pressures:					
	27 lb.	30 lb.	35 lb.	40 lb.	45 lb.	50 lb.
	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.	Minutes.
½ hour	.....	1	3	4	6	8
1 "	.....	1½	5	8	12	16
1½ hours	.....	2	6	12	18	24
2 "	.....	3	9	17	25	32
3 "	.....	5	15	25	34	45
4 "	.....	6	19	32	45	57
5 "	.....	8	22	37	50	61
6 "	.....	9	25	40	52	64
7 "	.....	10	27	41	54	65
8 "	.....	11	28	43	55	66

The times in Table 3, being for instant decompression, must be suitably extended for slow decompression.

Table 4 is taken from Fig. 7, and in part is confirmed by the experience in the East River tunnels as comparatively safe. Caisson disease will result, but no fatal or severe case should be experienced with physically sound men if these times are adhered to.

TABLE 4.—DECOMPRESSION TABLE BASED ON 9 MINUTES BEING SAFE FOR 27 POUNDS GAUGE PRESSURE.

Gauge pressure, in pounds.	Reduce pressure in 3 minutes to:	Total time in air-lock after 8 hours' work:	Total time in air-lock after 3 hours' work:	Total time in air-lock after 2 hours' work:
27	6 lb.	9	....	....
30	7½ "	24	....	....
32	8½ "	33	25	....
35	10 "	....	35	....
40	12½ "	....	48	....
42	13½ "	....	51	37
45	15 "	....	....	42
50	17½ "	....	....	48

Some time after reading Dr. Haldane's paper and studying his theory, it became necessary to raise the pressure in the tunnels to 40 lb., gauge. It was possible to make the workmen pass through three sets of air-locks on leaving the tunnel. The inner chamber was kept at 40 lb., the intermediate chamber at 29 lb., and the outer chamber at 12½ lb.

The men were ordered to take 5 min. in the first lock, 8 min. in the second lock, and 15 min. in the third. There was a distance of approximately 1 000 ft. between each pair of locks. Walking this distance and gathering in the stragglers generally required 10 min. to each chamber, so that, in all, 48 min. were taken to decompress from 40 lb. to atmosphere. No severe or fatal cases resulted, and little time was lost by the men through caisson disease, the cases being only slight. Under this pressure 330 men were employed for 36 days, working 3 hours on, 3 hours off, and 3 hours on. It is true that no green men were used on this work, as there were plenty of experienced air men available at that time.

The third diagram on Fig. 6 shows the desaturation curve and rate of decompression, leaving the saturation of the blood equivalent to 27 lb. on reaching atmospheric pressure. This result bears out in part the periods of decompression of Table 4, and, if a more rapid passage had been made through the first and second locks, probably a better result would have been attained, as the equivalent pressure of the blood would have reached 25 lb. on coming out, as shown by Fig. 7, instead of 27 lb.

In caissons or tunnels with but one lock, it is a difficult problem to allow workmen as long a time as Table 4 indicates, as no one can enter the air-lock during decompression. One method of overcoming this difficulty would be to provide a lock with two small end chambers

and a larger center chamber, four doors in all being necessary. Anyone making a short visit to the caisson could pass through without disturbing the pressure in the middle decompressing chamber. Fig. 8 is a diagram of such a lock with connections.

Some one has suggested, for diving bells and caissons, a detachable chamber like a boiler which the men could enter, and thus take as long as necessary to decompress.

AIR-LOCK WITH MIDDLE DECOMPRESSING CHAMBER

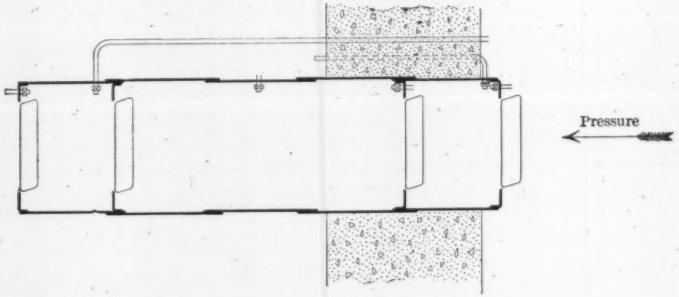

FIG. 8.

The men on the East River tunnels rebelled against 15 min. for decompression, but, after putting the responsibility up to the foremen, in time they found that it was a safeguard and voluntarily lengthened the time to 20 min., and gladly submitted to 48 min. for 40 lb.

The death rate due to caisson disease was comparatively small, averaging $\frac{19}{100}$ of 1% for the whole of the compressed-air work, and, from the experience gained, it would in all probability have been much higher if the decompression had not been lengthened.

The fact that the only recognized cure for caisson disease is recompression in a medical air-lock followed by slow decompression, is a powerful argument in favor of slow decompression, and where it is at all possible, in future works, regulated decompression will in all probability be adopted.

In caissons with small air-locks, the volume of air remaining when the lock is full of men is very slight, and very rapid decompression takes place. The workmen have a good opportunity to become seasoned in a caisson, as the pressure begins at 1 or 2 lb. and gradually increases day by day as the caisson sinks, and the highest pressures are required for but a few days.

On the East River tunnels two caissons were sunk to a final pressure of 33½ lb., and very few cases of caisson disease occurred, none of which was fatal, although the decompression was rapid. On the other hand, on account of the tunnels on the Manhattan side starting out at a high pressure, the men had no chance to get seasoned, and many cases occurred, though the time of decompression was regulated, but not to such an extent as Table 4 indicates.

Thus far, practically, no grandfatherly legislation hampers engineers in the United States in compressed-air work, but such legislation is threatened, and engineers should be prepared to guide it. The engineer is confronted with conflicting testimony on all sides, and, though his experience shows that some men are capable of rapid decompression without injury, he also knows that many men are injured. Because a few men can for a time rashly take the risk, shall he eliminate good men from compressed-air work by putting them to such a test without endeavoring to make it safe for all who are physically sound?

Prevention is better than cure, but absolute safety is impracticable, and some risk must be taken. In the writer's opinion, the hours of labor and time of decompression are matters that should receive the attention of engineers. The effect of legislation formulated by men ignorant of the conditions might greatly embarrass construction work, and, although it is contrary to the custom of this Society to draw up rules and regulations, yet it might render valuable aid if a committee of engineers, contractors, and doctors could be convened to study this subject and collect data, and ultimately draw up a table which would be a guide to the Profession. Such a table would be no doubt a compromise between the time worked in the air pressure and the time taken to decompress. It would be a statement of a body of men in high repute; if acted upon, it would relieve construction companies of liability to a great extent, and, if carefully formulated, it would greatly lessen the dangers of compressed-air work.

The writer is indebted to Alfred Noble, Past-President, Am. Soc. C. E., Chief Engineer of the East River Division of the Pennsylvania Tunnel and Terminal Railroad; and to E. W. Moir, M. Am. Soc. C. E. Vice-President of S. Pearson and Son, Inc., for permission to include such information as is taken from the work of the East River tunnels; and to Dr. Haldane for permission to make use of Table 1, and, generally, for the benefit of his theories, in writing this paper.

DISCUSSION

Dr. Haldane. DR. J. S. HALDANE, F. R. S. (by letter).—After very carefully reading Mr. Japp's paper, the writer considers it to be an extremely valuable contribution to a practical solution of the prevention of caisson disease. He clearly points out the difficulties involved in adopting without modification for all compressed-air work the plan of "stage decompression" worked out by Dr. Boycott, Lieutenant Damant, and the writer, which has been in successful use since 1907, for divers of the British Navy. A full account of the experimental evidence on which this plan is based will be found in the *Journal of Hygiene,* 1908, pp. 342-443. It will be seen from this paper that most of the experiments were made at very high pressures, such as 75 lb., and that they were carried out with a view to deep diving. It was for many reasons desirable to leave a good margin of safety; and, therefore, this margin was left, particularly as practical experience in deep diving was very limited, on account of other causes which the writer and his associates had just succeeded in removing.

Work in tunnels and caissons, however, is conducted at much lower pressures; practical experience is much more wide; and, thanks mainly to Mr. Moir, recompression is usually readily available. It becomes a question, therefore, whether the margin of safety which was allowed in decompression cannot be narrowed without appreciable danger. The new evidence which Mr. Japp has brought forward certainly seems to indicate that this is the case. He records the fact that in the construction of the East River Tunnels 330 "seasoned" men were employed for 36 days in two 3-hour shifts at a pressure of 40 lb., and were decompressed by a modified form of stage-decompression in 48 min., without a single serious case of caisson disease occurring. Thus there must have been about 24 000 individual decompressions without any mishap. On the principles laid down in the writer's paper, Mr. Japp calculates that the maximum air-saturation left in any part of the bodies of these men at the end of decompression, corresponded to a pressure of 27 lb. The writer's calculations agree with this estimate. In view of these results the author proposes to shorten stage-decompression in such a way that in place of the 19 lb. of residual saturation which the writer has proposed, 25 lb. should be left in the case of men who are "seasoned" to the work. This would greatly shorten stage-decompression, and render it a much more easily practicable process, although the pains in joints, etc., which are the most easily produced and least serious symptoms of caisson disease, would not be prevented.

The reason for the proposed 19 lb. is that occasional serious cases seem to occur at working pressures down to 20 lb., and that a fatal case is recorded at 23 lb. On the other hand, there can be little

doubt that for the great majority of men a pressure of 25 lb. would not cause appreciable danger. Provided, therefore, that susceptible men can be excluded, Mr. Japp's proposal would seem to be consistent with reasonable safety. Dr. Haldane.

Susceptible individuals can doubtless be excluded, to some extent, by a careful medical examination, with special regard to the state of the heart and blood vessels, and to any general debilitating cause, among which age must be included. Another very important factor is fatness. Dr. Boycott and Lieutenant Damant* have recently shown by experiments on animals that when the exposure to compressed air is prolonged, fatness increases the susceptibility to caisson disease. They decompressed 70 guinea pigs at a dangerous rate after exposure to a pressure of 100 lb. Exactly half of the animals died of caisson disease, and the other half were killed. The percentage of fat was then determined in the bodies of the two sets of animals, and was found to be much greater in those which had died of caisson disease, the average difference being in the proportion of 225 to 100. The caisson disease simply picked out the fatter animals. In consequence of these experiments, which confirmed in a most striking manner the conclusions which had been previously reached on theoretical grounds, all British naval divers are now disqualified at the medical examinations from deep diving if they are inclined to fatness.

While a medical examination will probably exclude most of the more susceptible individuals, it still seems very doubtful whether all can be excluded in this way. With new men, therefore, it is desirable that preliminary trials should be made with short shifts, as was the practice in the East River Tunnel work. These men should be carefully watched after decompression, and promptly recompressed if they show any threatening symptom; and any man who has shown threatening symptoms should be rejected.

With these precautions, in addition to having a medical air-lock and keeping the men within reach of recompression for about an hour after they come out, the writer thinks that in all probability Mr. Japp's plan would prove successful. For casual visitors, the duration of stay could be limited, so that on coming out the maximum saturation in their bodies should not exceed 19 lb.

The conditions vary so widely in different kinds of work in compressed air that it seems hardly desirable to lay down by legislation or otherwise any hard-and-fast general rules as to decompression. The writer, however, thinks that for any particular undertaking in compressed air, special rules, suitable to the particular circumstances, should be drawn up and strictly enforced. If these rules could also have some authoritative sanction, as, for instance, in the case of

* *Journal of Hygiene*, 1908, p. 445.

Dr. Haldane. "Special Rules" under the Coal Mines Regulation Act in England, engineers and contractors would be relieved to a large extent of what at present must be a very uncomfortable responsibility.

Dr. Keays. DR. FREDERICK L. KEAYS* (by letter).—The prevention of compressed-air illness is, of course, most important. It is likely that with the modern demands of engineering, even greater compressed-air undertakings will be projected in the future than in the past. Any means, therefore, which will add to the safety of the workmen, will not only benefit humanity at large, but will also permit of more extensive undertakings.

The three great factors in the causation of compressed-air illness are: The degree of pressure, the time under pressure, and the decompression period. The degree of pressure is, of course, the chief factor; and it may be stated that, other factors remaining the same, the higher the pressure, the greater will be the dangers of compressed-air illness. The degree of pressure which is to be used in any undertaking is not a matter of choice, but depends on the depth to which the work must be carried. The only means of preventing illness, then, which are left to the engineer are the regulations of shifts and of decompression periods. In the past it has been thought that the longer one stays under pressure, the greater will be the dangers of compressed-air illness. This is true, up to a certain point, namely, that at which there is complete saturation of the blood and body fluids with the gases of the air. Dr. Haldane has made a valuable contribution to the subject of compressed-air illness by his work on the question of body saturation, and Mr. Japp has worked out Dr. Haldane's theory in a plain and satisfactory manner.

The body fluids are practically saturated at any given pressure in about three hours. This belief is based on the results of the work in the Pennsylvania East River Tunnels. From a study of the records, it was found that men working for periods of less than 3 hours were less likely to have "bends," and much less likely to have a serious illness than those who worked for 3 hours or longer. Furthermore, it was found that in comparing the results of 6-hour shifts—which, since they were divided into two 3-hour periods with an interval of 3 hours at normal pressure, really meant 3 hours' exposure—with the results of 8-hour shifts, at almost equal pressures and with corresponding rates of decompression, there were fully as many cases of illness with the former shifts as with the latter, and certainly more serious and fatal cases. A point which has not been sufficiently emphasized in the past is that the real danger in compressed-air work comes with or after decompression. For this reason the practice, which has been almost universally followed, of dividing the working shift into two periods,

* Late Medical Director for S. Pearson and Son.

is a bad one, for it doubles the number of decompressions. The results would be better in tunnel and caisson work, if continuous shifts of 4, 6, or 8 hours were arranged to suit the pressures; say, 4 hours for pressures of from 42 to 50 lb. +, 6 hours for pressures of from 32 to 42 lb. +, and 8 hours for pressures below 32 lb. +, and if the decompression periods were increased to reasonably safe lengths of time. Dr. Keays.

The question of the decompression period is a difficult one to settle. In the first place it must be admitted that certain men have individual susceptibility to the effects of compressed air. Just what the reason for this susceptibility is has never been learned. The fact remains, however, that while one man may work even at a high pressure and decompress rapidly without any symptoms of illness, another, under even more favorable conditions of pressure, length of shift, and time of decompression, may succumb to sudden death from syncope and collapse, and the autopsy will show large quantities of free gas in the circulatory system. Among practical compressed-air workers there is a decided difference of opinion as to the necessary length of decompression periods, some even claiming that quick decompression is preferable. Nearly all men, however, who have dealt with the subject from the physiological and medical standpoint, have advocated fairly long times for decompression. The writer believes that lengthening the time of decompression is the most efficient means of reducing the number of cases of compressed-air illness, as well as the number of serious cases.

On account of the common occurrence of cases of delayed onset of symptoms, it is not believed that lengthening the decompression to periods claimed by Bert, von Schrötter, Hill, and Haldane, to be safe, would prevent all cases of illness, or even serious and fatal cases. In the writer's experience a comparatively large number of men first begin to have symptoms of compressed-air illness several hours after decompression, in some cases after as many as 12 or 15 hours, and in a fairly large number, after 3 hours. In two fatal cases symptoms began to appear between 2 and 3 hours after decompression. Why these men should not have had symptoms, even if they had taken as long as 1 hour for decompression, the writer cannot see. He believes, however, that reasonably long decompression periods are efficient in reducing the number of cases of illness, disability, and death.

Just what are reasonably safe lengths for decompression periods have never been determined. In the work on the Pennsylvania East River Tunnels after April, 1906, a rate of decompression of at least 1 min. for each 2 lb. pressure was required, and quite generally followed. This seemed to cause less illness and a reduction in the number of serious cases, as compared with previous months when there were no rules as to the rate of decompression. Even this rate of decompression, which was longer than has been required in any compressed-

Dr. Keays.

air work in the United States, did not prevent serious and fatal cases. For this reason the writer thinks that even slower rates of decompression should be tried, for pressures above 28 lb. +, below which, this rate of 2 lb. per min. appears to be safe.

Mr. Japp has set forth Dr. Haldane's method of "stage" decompression, which is claimed by the latter to be more efficient in preventing illness than uniform rates of decompression of even longer times. The writer does not think that enough experimental work has been done to prove the efficiency of "stage" decompression; but its use in diving has proved very satisfactory, and experience with the extra bulkhead in tunnel work, which necessitates an interrupted decompression roughly corresponding to Dr. Haldane's method, would argue favorably for "stage" decompression.

If it were possible, in accordance with Dr. Haldane's suggestion, to arrange a large decompression chamber in which a pressure equal to one-half the actual tunnel pressure could be maintained, where the men could pass a long decompression period in comfort, and use the time for bathing and dressing and, perhaps, for eating, it would probably prove a valuable means of preventing illness, and, at the same time, permit of working with comparative safety under higher pressures than could now be attempted.

Medical supervision and examinations are important and are instrumental in preventing serious results from the effects of compressed air. Men with organic disease are certainly less able to withstand the effects of compressed air than normal individuals. New men are certainly more susceptible to the effects of compressed air than old workmen. When high pressures are being used, a preliminary test of about 1½ hours under tunnel pressure for all new men, followed by a second medical examination, and short shifts of from 1½ to 2 hours for the first working period, are effectual means of guarding against serious results among new workmen. These means proved very valuable in the work on the Pennsylvania East River Tunnels.

Much remains to be learned in regard to the prevention of compressed-air illness. This knowledge can be gained only by experimental work with animals, supplemented by records of compressed-air undertakings. By adopting new methods which appear to be improvements upon the old, we may hope to arrive, finally, at a more definite knowledge of what is reasonably safe in the lengths of shifts and decompression periods.

Compressed-air work in tunnels and caissons, where pressures above 30 lb. + are used, will always remain a dangerous occupation in spite of all reasonable precautions, for one can hardly hope to prevent all cases of illness. Those cases due to small quantities of free gas are almost sure to happen, and a small proportion of them is likely to result in paralysis, which may cause permanent disability or death.

Until the reason why certain men are susceptible to the effects of compressed air, can be learned, and such individuals can be recognized, cases of sudden death from large accumulations of free gas are almost certain to occur occasionally. Dr. Keays.

Dr. Albert J. Loomis.*—The author deserves great praise for the clear and lucid manner in which he has stated the ideas of those who have investigated and written on the subject of compressed-air diseases. Dr. Loomis.

But, after a pretty wide experience, extending back over twenty years or more, with pressures ranging from 15 to 42 lb., gauge pressure, the speaker cannot agree with him, or his authorities, as to the cause of the disease under discussion. While admitting the "saturation theory" as laid down by Bert, von Schrötter, Haldane, and others, the speaker does not think that it is the cause of the so-called caisson disease, for the following reasons:

First.—As to the formation of bubbles of gas or air in the blood-vessels themselves: They have only been demonstrated in exceedingly high pressures—15 or 20 atmospheres—which, at the present time, are considered impracticable as working pressures. If these bubbles should occur in the blood-vessels of the men, working as they do and coming out as they do, there would never be any use for hospital locks, as the men would all die of suffocation before they could be gotten to them, owing to these minute bubbles becoming entrapped in the very small vessels or capillaries of the lungs, and thereby preventing the oxygenation of the blood, a condition which operating surgeons frequently meet, to their great regret.

Second.—If the symptoms were due to any change in the blood itself, there would be a general set attacking all parts of the body to a greater or lesser extent, which is the very thing that does not happen, but, on the contrary, they show themselves in a very few localities, and most of these localities are in the lower extremities, situations which any surgeon who is at all proficient in locating nervous lesions by their symptoms, would place without hesitation in the spinal cord, and in the lower portion of the cord.

On the other hand, there is practically but one set of symptoms, the so-called "staggers," which would lead physicians to look for a cause or lesion in the brain, a condition which is quite rare when compared with the great number of cases occurring in the lower extremities.

Should there be any change in the composition or condition of the blood, the brain would naturally be the first organ to show such change, as it is without doubt the most delicate organ of the whole anatomy.

In reference to the CO_2, the engineers of the North River Division of the Pennsylvania Tunnels have prepared a chart on which all the

* Chief Medical Officer, North River Division, Pennsylvania Tunnels and Hudson Companies.

Dr. Loomis. cases of compressed-air sickness have been plotted, and this chart shows that, on that work, the greatest number of cases occurred at a time when the CO_2 was only 1 part in 1 000, with a pressure of from 20 to 25 lb.; and, likewise, when the pressure was 28 lb. and the CO_2 was 2 parts in 1 000, there were no cases of caisson disease.

While the speaker is not prepared to place much stress on the CO_2, other than as an indication of the general ventilation, he does think that weather changes play a very great part in the causation of the disease, as do also the habits of living of those engaged in the work.

When it comes to the examination of men, the medical officer should fit himself for the work by a close study of the conditions under which the men are to work, and his power of rejection should be absolute. The selection of the men should be the duty of one man who should be responsible for their welfare while they are at work under pressure.

In substantiation of these opinions, the speaker desires to state that, in the work of the Hudson Companies, in the construction of the Hudson and Manhattan Tunnels, about 38 000 men have been examined, since 1902, for pressures of from 15 to 42 lb., with three deaths, and in the construction of the North River tubes of the Pennsylvania Tunnels about 7 000 men have been examined for pressures of 15 to 38 lb., with no deaths.

Mr. Saunders. WILLIAM L. SAUNDERS, M. AM. SOC. C. E.—This is a subject about which very little of practical value has been written, and the speaker is glad to note that Mr. Japp has gone into the matter in so much detail, both from theoretical and practical standpoints.

Caisson disease, or the "bends" as it is commonly called, does not differ from other diseases, in that doctors are very apt to disagree about it. Heretofore the subject has been treated in print mainly from a medical standpoint, various theories being advanced as to the cause of this disease, that is, whether it is due to mechanical or chemical action. The speaker remembers when the most plausible theory advanced for the cause of caisson disease was that the blood, which had been compressed and held in a state of tension, was suddenly released by decompression, creating a pain when coursing through the veins. This was illustrated by the statement that if a string is wrapped tightly around one finger and then suddenly unwound, the finger is temporarily in pain. The subject has been discussed so fully by Mr. Japp, however, and the theory of caisson disease has been so well established by the experiments to which he refers that it is now generally accepted that bubbles are the cause of the "bends."

Compressed air is the speaker's hobby, and in his early experience in seeking information about it he has soared, like the eagle mentioned by Mr. Japp, to a height of 3½ miles, where he has felt the earmarks of the "bends," and has descended to great depths in mines and in a

diving apparatus under the sea, where similar sensations have been felt. Mr. Saunders.

Personal experience in a matter of this kind always fits one to judge better the causes as well as the effects, and it may add something to the interest in this matter to recite an experience which the speaker had some fourteen years ago in the old Hudson tunnel, now called the McAdoo tunnel. This tunnel was started by Mr. D. C. Haskin a little before, or about the time of, the Centennial Exposition in Philadelphia, in 1876. Mr. Haskin conceived the idea of driving a tunnel under the Hudson River by maintaining a chamber of compressed air to hold up the face. He used no shield, and as the material through which he worked was more or less unstable, he met with considerable difficulty, and some lives were lost. He sank his entire fortune in the enterprise, the work being taken up later by English capitalists, who contracted with Messrs. Pearson and Son to finish the tunnel. At that time E. W. Moir, M. Am. Soc. C. E., was in charge of the work, and the experience which he gained there has no doubt helped him to design and carry through the recent tunnel construction under the East River, which Mr. Japp has so ably referred to and with which he has been connected in so important a position. During the administration of the Pearsons, the late R. P. Rothwell, M. Am. Soc. C. E., at that time editor of the *Engineering and Mining Journal,* and the speaker were invited to examine the tunnel for the purpose of making a report thereon. About an hour was spent under some 35 lb. absolute pressure. Coming out of the locks rather rapidly, on reaching the washroom, the speaker was taken with a severe attack of the "bends," the effect being like that of a paralysis of the lungs. A horizontal tank, resembling a common horizontal boiler, was at hand, and the patient was transferred to a bench of straw, the door was closed, compressed air was turned on, and the relief was instantaneous. A pressure valve, provided for the purpose of admitting and discharging pressure from the tank, was used to enable the prisoner to escape, but it was noticed that when the pressure fell only two or three points the "bends" returned, and it was only by the gradual expansion of the pressure, occupying about ½ hour, that the patient was able to open the door and walk out cured. The necessity for reducing the pressure gradually, as shown very graphically by the return of the "bends," indicates the value of the theory of gradual decompression. It also bears out the idea that time must be given to permit the escape of the little bubbles which accumulate in the body.

An interesting incident was noted in connection with this experience. When the pressure in the tank was lowered, through the discharge of air, a fog formed instantaneously, and as soon as air was let in and the pressure increased, this fog was as promptly dissipated. The speaker experimented with this for some time—manufacturing and annihilating fog. The lowering of pressure, of course, resulted in

Mr. Saunders. cooling the air, and, *vice versa*, increasing the pressure heated the air. As the air was cooled, fog was precipitated immediately when the temperature reached the dew point, the capacity of air for moisture depending entirely on volume and temperature, that is, a certain fixed volume of compressed air or free air can hold only a certain quantity of moisture at a certain temperature. If the temperature is raised, it will hold more water; if the temperature is lowered, the moisture must be precipitated first in fog, and later, in the form of rain.

Mr. Aims. WALTON I. AIMS, M. AM. SOC. C. E.—The literature on caisson disease is quite extensive. The causes of the disease now seem to be well determined, but difficulty is found in conforming practically with the medical rules and regulations for its prevention.

The age of the compressed-air worker has an important bearing on his ability to withstand the effects of too rapid decompression, but youth generally lacks experience, and, in difficult work, some compromise has to be made in drawing the age limit for entering the compression chamber. Dissipation acts strongly in undermining a man's resistance to the effects of compressed air, especially among workers of mature years, and even some temporary indisposition may occasion an attack in one who has previously been pronounced physically fit by a medical examiner.

In nearly all compressed-air work there is difficulty in compelling the workmen to take a reasonable time in the air-lock while decompressing. Believing that this is due largely to the extremely uncomfortable conditions in the ordinary air-lock, a method for relieving the lock of the cold and fog when decompressing was used at the Battery-Brooklyn tunnel by introducing a constant flow of heated dry air while the lock was being exhausted. In addition to removing the cold and fog from the lock, this heated air also provided for ventilation while decompressing, a feature not found in the ordinary compressed-air lock. The comparative freedom from caisson disease at the Battery-Brooklyn tunnel, where practically no age limit was established, seemed to indicate the value of thus heating and ventilating the air-lock during the decompression period.

Dr. Erdman. DR. SEWARD ERDMAN.*—The speaker wishes to acknowledge his appreciation of Mr. Japp's instructive contribution to the literature of caisson disease, and feels that the general adoption of the suggested "Step Decompression Method" will aid much in robbing compressed-air work of its gravest dangers.

The reduction, to terms and curves of mathematical precision, of the time and pressure elements, as shown on his charts, is most interesting; but it seems to the speaker that there will always be another factor to be reckoned with, one which cannot be reduced to

* Formerly a Medical Officer on the East River Tunnels Contract.

mathematical terms, and yet which will continue to render uncertain the result of exposure to compressed air in each individual case. Dr. Erdman.

This indeterminate factor may be called the "personal equation," and seems to be dependent on the functional efficiency of the circulatory and respiratory apparatus.

In the experience of all who have seen much of compressed-air work, cases are not infrequent where men, who are seasoned air workers, will work daily, for long periods of time, at fixed conditions of pressure, length of shift, and decompression time, without suffering any ill effects, but who come to work the next day, under exactly the same conditions, only to develop a severe or even fatal attack of "bends."

The decompression time, indicated on Mr. Japp's charts, can be controlled by valves from the air locks; but, for the individual, desaturation of the body tissues and the blood must take place through the lungs; therefore the lungs act as the "escape valve" for the individual.

In explanation of such accidents as are above cited, it is likely that this "escape valve" is temporarily clogged.

Desaturation of any given part of the body will be impeded by causes which slow the blood current in that area.

Thus passive congestion of the lungs, or the presence of mucus in the air spaces (as from a cold) may act as clogs to the "escape valve," thus increasing the time necessary for desaturation, and, under such conditions, the body does not rid itself of the gases in the average time, and an attack of "bends" results.

T. Kennard Thomson, M. Am. Soc. C. E. (by letter).—The writer has read this valuable paper with much interest, and notes that the author does not refer to the old-fashioned remedy—the electric battery—probably because it is not nearly as reliable as recompression. The writer once cured himself of an attack of bends by electric shocks, after he had failed to get any relief from recompression; and the next time the battery failed to cure him and recompression did—and so it goes. He has had the bends after being under an air pressure of 23 lb., and was not affected by a pressure of 35 lb. on the same job. The purity of the atmosphere and the physical condition of the man at that particular time have much to do with the case. Mr. Thomson.

The writer, having no ear drums, has never had much trouble in keeping his ears from getting blocked, and, as a rule, he has been affected much less by the air pressure in vertical caissons than in tunnels; this he ascribes partly to the fact that the air in caissons is renewed more frequently than in tunnels and therefore is fresher, and partly to the increased time he was obliged to remain in the tunnels to get to the headings.

Under the Harlem River, when under comparatively light pressure, while sinking through the foul river bottom, his men have been much

Mr. Thomson. troubled by the bends, but have experienced very little inconvenience under much greater pressure while the caissons were sinking through the clean clay below the river-bed.

In an emergency, the writer has seen men work for 18 or 20 hours in pressures up to 25 lb. without any ill effects. He has gone in and out of the caissons repeatedly for 86 hours (without going to bed), under pressures up to 35 lb., without inconvenience—but would never be guilty of such hours again.

When he had the bends, the pain disappeared while he was in a perspiration induced by drinking hot coffee, only to return when he had cooled off. He then tried a very hot bath and felt well until he had dressed.

As stated by Mr. Japp, slow decompression is the best preventive known, and recompression is not always reliable. As the author also states, hot coffee should always be taken on coming out.

Mr. Japp. HENRY JAPP, M. AM. SOC. C. E. (by letter).—The discussion, thus far, seems to indicate that the general opinion is that the cause of caisson disease is the presence of gas bubbles in the blood and tissues, the only one taking exception to this being Dr. Loomis, who states that bubbles have only been demonstrated under exceedingly high pressures—15 or 20 atmospheres.

The writer cannot agree to this, as he has seen the results of three autopsies on men who have died on coming out of pressures between 30 and 35 lb. gauge, and in each case bubbles were plainly visible to the naked eye in the blood vessels; and he has read reports of many more, and in almost every case similar bubbles have been seen. In one case Drs. McWhorter and Erdman were able, at the autopsy, to collect and analyze the air which escaped from a large bubble in the heart. It may be thought that these bubbles are not present during the life of the subjects, but the writer has also seen what had every appearance of being little bubbles passing along in the blood vessels of the wrist of a member of the East River Tunnels staff about half an hour after he came out of the air pressure.

Dr. Loomis' extraordinary success, in having so few deaths as he mentions, is highly commendable, but there is no doubt that if a larger number of fatal cases had passed through his hands he would have made similar observations.

The fact that Dr. Leonard Hill has exhibited the formation of bubbles and the stoppage of the circulation in the web of a living frog by sudden decompression, and the disappearance of the bubbles and the resumption of the circulation by recompression, should of itself show that, although this was obtained by high pressures for the purpose of rapid demonstration to an audience, the same thing can readily occur with a lower pressure.

Mr. Japp.

While Dr. Loomis does not accept the theory that bubbles of gas cause bends, he presents no alternative.

The writer is much indebted to Dr. Haldane for checking the calculation for stage decompression from 40 lb. pressure, and is glad to have it confirmed.

Dr. Haldane suggests that as the conditions vary so widely in different kinds of work, it is undesirable to lay down by legislation or otherwise any hard-and-fast rules as to decompression, and in the paper the writer calls attention to the danger of legislation if it is not guided by the Profession.

Since this paper was written, however, the New York State Legislature has brought forward an act, now signed by the Governor, and taking effect January 1st, 1910, which limits everything in a very hard-and-fast manner. The hours of labor and the rates of decompression are regulated according to Table 5.

TABLE 5.

Gauge pressure, in pounds.	Time under pressure.	Interval between spells.	Uniform decompression, in minutes.
0–28.....	8 hours, less interval.....	30 consecutive minutes spent in the open air.....	18¾
28–35.99....	2 spells of 3 hours each...	At least 1 hour.	24
36–41.99...	2 spells of 2 hours each...	At least 2 hours.....	42
42–45.99....	2 spells of 1½ hours each.	At least 3 hours.....	46
46–49.99....	2 spells of 1 hour each....	4 hours.....	50

No employee shall be permitted to work in pressures exceeding 50 lb. per sq in. except in cases of emergency.

The decompression shall be at the rate of 3 lb. every 2 min., unless the pressure shall be over 36 lb., in which event the decompression shall be at the rate of 1 lb. per min.

This law is a great step in advance of anything attempted privately in the United States, and the Commissioner of Labor is to be congratulated, but if these rates are compared with the suggested Table 4 of the paper, it will be found that the new law proposes a greater degree of safety for the lower than for the higher pressures, and inasmuch as the law requires that the decompression shall be uniform, the final pressure in the blood on coming into the atmosphere will be much higher than would result from stage decompression.

In order that this may be seen more clearly, the writer has calculated the pressure in the blood for various pressures, decompressing uniformly as required by the law, and also by stage decompression, and gives the results in Table 6.

It will be noted that, whereas the stage decompression proposed fixes the pressure in the blood on emerging from the air-lock at a constant of 25 lb. per sq. in., the uniform decompression periods required by the law give pressures in the blood on emerging varying

Mr. Japp. from 25.70 up to 32.50 lb., such a result being obtained in the case of 50 lb. with 17 min. longer for uniform decompression than stage decompression, and when it is noted that, for a pressure as low as 9 lb. per sq. in., the law requires 2 min. for every 3 lb., or 6 min. for decompression, it will be seen how inconsistent are the requirements. In other words, the new law requires generally more time for uniform decompression than is needed for stage decompression, while giving less safety.

TABLE 6.

Tunnel pressure, in pounds (gauge).	Time worked, in hours.	Uniform decompression, in minutes.	Pressure in blood on emerging, in pounds.	Stage decompression, in minutes.	Pressure in blood on emerging, in pounds.
28	8	18⅔	25.70	14	25
36	3	24	30.25	36	25
41.99	2	42	31.25	37	25
45.99	1½	46	32.00	35	25
50	1	50	32.50	33	25

It is doubtful whether Dr. Keays is correct in stating that the body fluids are practically saturated at any given pressure in about 3 hours. Fig. 6 shows that in 3 hours the slower saturated parts are only 81% saturated, but the writer is of the opinion that much could be done toward ascertaining the rate of saturation by further experiments.

With regard to Dr. Keays' remarks on "delayed caisson disease" (that is, cases which occur many hours after decompression), and his deduction, from those which occurred after too rapid decompression, that the lengthening of the decompression to periods suggested by Dr. Haldane would not prevent all cases of illness, or even serious and fatal ones, this cannot be accepted as more than an expression of opinion, inasmuch as no cases had occurred in Dr. Haldane's experience when his rules were adopted, and it is doubtful if any delayed cases would occur or would have occurred had Dr. Haldane's rules been adopted, as the sole object of slow-stage decompression is to prevent the formation of injurious bubbles.

Delayed caisson disease after too rapid decompression can be explained by the accumulation of gas in tissues, which, though strong enough to stand the pressure of the accumulated gas for some hours, ultimately break down under the strain. Liberated gas collected in pockets cannot very readily be absorbed again excepting by recompression.

Mr. Saunders' description of the effects of recompression and decompression in his own case, when he was paralyzed, is an excellent illustration of the bubble theory.

Dr. Erdman's idea of the lungs acting as an escape valve is very ingenious, and is quite in line with the general mechanical theory of

caisson disease. An investigation on the lines of his suggestion would be very valuable. Mr. Japp.

Mr. Aims' reference to the effect of dissipation on the resistance of men to the effects of compressed air and to the effect of the general comfort of the men coming out of the air-lock is generally accepted as important, but with slow decompression the discomforts of fog and cold are not felt in air-locks, and therefore heating is not as important as in the case of more rapid decompression.

Dr. Loomis' suggestion that the weather changes play a very great part in the causation of disease is not borne out by observations made by the medical staff on the East River Tunnels.

Mr. T. Kennard Thomson's remarks are valuable, coming from a man with so much actual experience in air pressure, and his acceptance of slow decompression as being the best preventive known—coming as it does from an engineer, the bulk of whose experience has been in caissons with small air-locks, where slow decompression is not an easy matter—leads one to hope that, even under such disadvantages, practical men will ultimately arrange for a slow decompression.

The reason why recompression is not always reliable, as Mr. Thomson says, is that the bubbles having torn the tissues and done the damage, no amount of recompression which merely removes the bubbles will heal the torn tissue, but if recompression can be resorted to promptly, before any permanent injury is done, then relief will be obtained.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1119

COPYRIGHT IN DRAWINGS OF A TECHNICAL CHARACTER.

By D. A. Usina, Assoc. Am. Soc. C. E.

The copyright law, enacted March 4th, 1909, to go into effect July 1st, 1909, contains provisions which may be of great importance to engineers, in that it provides specifically for copyright on "Drawings (or plastic works) of a scientific or technical character," and in that the securing of the copyright is made an extremely simple matter.

Prior to the passing of this law there has been no certain protection against the pirating of designs for engineering structures. Where a design though new has involved only the skill of the engineer, it has been unpatentable, and, unless the work could be classed under the fine arts, there has been no reasonable hope of protection by copyright. In many cases designs involving a high degree of engineering skill and gotten up at great expense have been copied by others with no moral justification whatever. National, State and city governments and private parties have asked for bids to be accompanied by original designs, and have appropriated the design of one bidder and let the contract to another—a species of piracy distinguished legally but not morally from the piracy of works of literature or the fine arts.

It is hoped that the new law will put a material check on such practices by reserving to the proprietor of such drawings the sole right of printing or copying them. It is doubted if the law can be invoked to prevent the building of the structures shown in such drawings, with the possible exception of works which might be classed in

the fine arts. The limitations which can be put upon the use of the drawings, however, should afford a substantial degree of protection, and the ease with which copyright can be secured makes it undoubtedly worth trying for whatever protection can be secured.

The language of the Act relative to technical drawings is as follows:

"That the application for registration shall specify to which of the following classes the work in which copyright is claimed belongs: * * * (*i*) Drawings or plastic works of a scientific or technical character."

Taken in connection with the remainder of the law, there is little or no doubt that this language was intended to render such drawings copyrightable. The view of the Register of Copyrights is:

"that the language of the Act would authorize the deposit and registration in this office for copyright protection of drawings of a scientific or technical character such as under the present law would not be included in the term 'drawing,' which seems to be confined to artistic drawings. Examples of such works would be drawings for machinery, for engineering construction, for architectural works, etc. While these drawings have to some extent been registered in the Copyright Office heretofore, it has always been a question whether the law authorized the registration and whether copyright protection would follow."

Protection is extended to "all the copyrightable component parts of the work," such for example as the drawings relating to any complete and mentally segregable part of the whole structure.

Copyright is secured by publication with the required notice affixed to each copy; or, where "not reproduced for sale," by depositing a photograph or other reproduction with the Register of Copyrights. But engineering drawings are usually offered for sale (as part of the bidder's work), and, in such cases, the copyright is secured by affixing a notice of copyright; and after thus securing the copyright there must be promptly deposited in the Copyright Office two complete copies.

Several permissible forms of notice are provided. It is sufficient if the word "Copyright" be applied to each sheet of drawings, together with the name of the proprietor and the date. It is customary now to apply the name and date to drawings; and the addition of the word "Copyright" is the simplest matter in the world. For what it is worth (and apparently it will sometimes be worth much) the word "Copyright" should be added to all original drawings and copies thereof.

In the case of drawings of especial value, it is advisable also to deposit copies promptly with the Register of Copyrights and to secure a certificate of such deposit, so as to obtain the apparent sanction of the Government, and to indicate such fact on the face of the drawings. For drawings of less value, the depositing of copies may be delayed until it is thought necessary to threaten or bring suit for infringement. The securing of copyright is probably effected by the mere notice. The deposit of copies is an additional formality which, though required to be made "promptly," and though a necessary preliminary step before a suit for infringement can be brought, does not seem necessary to the securing of the right.

Copyrighted works may be sold or furnished to others with any reasonable limitation upon their use. For example, a bidder may furnish an original design with a notice of copyright and a special notice that the copyright and the copies submitted constitute parts of the bid and remain the property of the bidder in case his bid is not accepted, and that the design is not to be copied or submitted for bids from others.

An interesting feature of the new law is the apparent sanctioning of actions against the United States and its officers; but much weight cannot be placed on this construction, in view of the broad prohibition in the general law against actions "sounding in tort" against the United States. Congress has provided no remedy for wrongs committed by the National Government, except for violations of contract, express or implied. The provision of the new law bearing on this subject relates specifically to the recovery of costs in suits for infringement and states that in such suits,

"except when brought by or against the U. S. or any officer thereof, full costs shall be allowed, and the court may award to the prevailing party a reasonable attorney's fee."

It is unnecessary to consider the question here at length, but it may be stated generally that after a tort is committed by the United States there is no adequate remedy, but an injunction may be secured against a repetition of it; and perhaps, if one has sufficient notice of an intention to infringe the copyright, such infringement can be enjoined in the first instance. Possibly the framers of the law had such injunctive proceedings in view when they framed the above

section exempting the United States and its officers from the payment of costs.

The Act provides also that "any person who willfully and for profit" infringes a copyright may be punished by a fine and imprisonment; and an infringer who had received such a special notice as is above referred to would be infringing willfully and for profit, and would hesitate to run the danger of fine and imprisonment.

Under the old copyright law there were only a few cases in which it was attempted to secure a copyright upon a purely technical drawing. It was necessary to enter the drawings in the office of the Librarian of Congress, and the Register of Copyrights permitted such entry only under protest and was of the opinion that such drawings were not copyrightable. The validity of the copyright was generally doubted. The new law, which specifies such works particularly as one of the classes in which copyright may be claimed, and permits the securing of the copyright by applying to the drawings a notice that copyright is claimed, is a material advance toward the protection of a kind of intellectual property of far greater dignity and usefulness than circus posters and similar matters, to which the previous law had extended protection. To quote the language of the Constitution, upon which is based the power of Congress to make a copyright law, technical drawings do more "to promote the progress of science and useful arts" than many of the classes of work heretofore provided for.

The practical value of the new law to engineers cannot be predicted. We can only hope. It cannot be used to protect functional equivalents of the thing represented by the drawings. That is the purpose of the patent law, and protection thereunder runs for seventeen years only, while copyrights may last fifty-six years. It is quite possible for the courts to construe the new law so narrowly as to destroy its value. But it will probably be construed so that it will protect the engineer against the copying by others of the substance or of any material part of his original designs.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1120

THE SIXTH STREET VIADUCT, KANSAS CITY.*

By E. E. Howard, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Daniel Bontecou, Victor H. Cochrane, O. E. Mogensen, N. T. Blackburn, George H. Pegram, and E. E. Howard.

The Sixth Street Viaduct, recently built, is a steel structure, about 8 000 ft. long, providing an elevated roadway and two car tracks across the broad, flat valley of the Kaw River to the high land on each side. It was designed to accommodate, and, by improved conditions for comfort, safety, and speed, to facilitate street-car and vehicular traffic; thus contributing to the unification of the social and commercial interests of Kansas City, Missouri, and Kansas City, Kansas.

The main viaduct and those portions of the side laterals or approaches now finished include, in general figures, 1 900 lin. ft. of roadway 38 ft. wide, 6 100 lin. ft. 30 ft. wide, 1 000 lin. ft. 24 ft. wide, and 1 000 lin. ft. 20 ft. wide, a total of 10 000 lin. ft. of roadway; also 8 500 lin. ft. of double-track railway. In this construction there have been used more than 120 000 lin. ft. of piles, 23 000 cu. yd. of mass concrete, 13 000 tons of steel, nearly 10 000 cu. yd. of reinforced concrete, 34 000 sq. yd. of asphalt pavement, besides minor and miscellaneous material. On one lateral, or side approach, where an incline could not be conveniently placed, there were installed two large electric traffic elevators of sufficient capacity to raise any vehicle and team. Figs. 1 and 2, Plate II, are general views of the viaduct and the bridge.

* Presented at the meeting of April 7th, 1909.

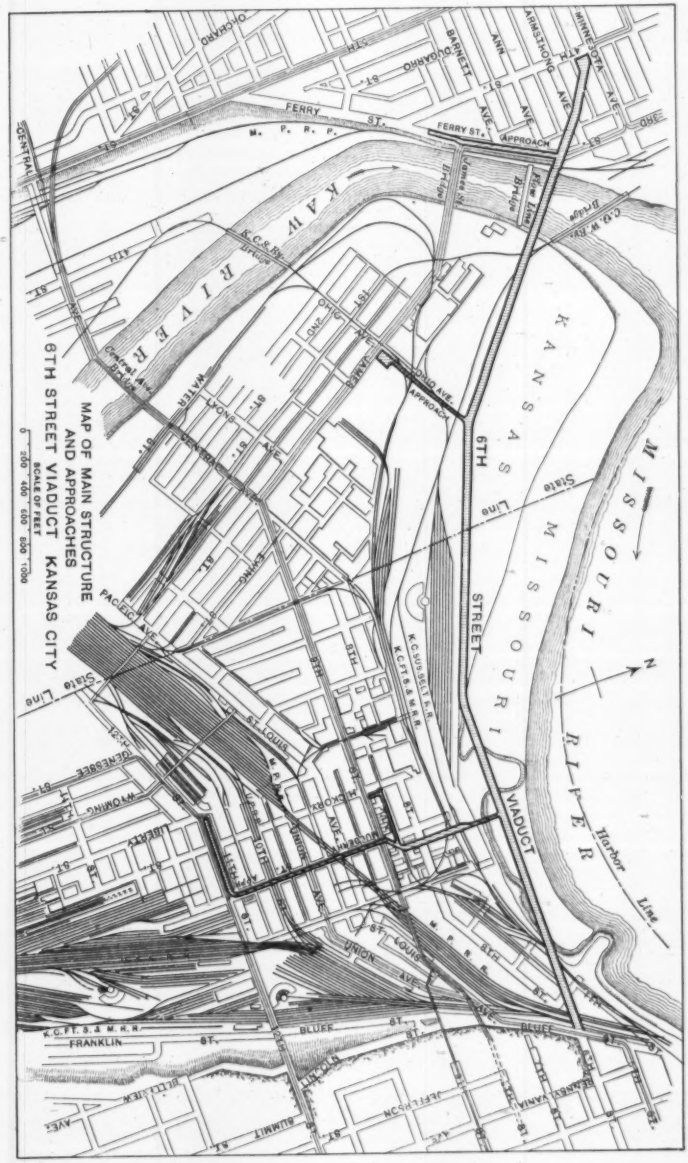

MAP OF MAIN STRUCTURE
AND APPROACHES
6TH STREET VIADUCT KANSAS CITY
SCALE OF FEET
0 200 400 600 800 1000
KANSAS MISSOURI
MISSOURI RIVER
6TH STREET VIADUCT
KAW RIVER
FERRY ST.
FERRY ST. APPROACH
M. P. R. R.
K.C.S.Ry. Bridge
Central Ave. Bridge
OHIO AVE. APPROACH
State Line
Harbor Line
N
PACIFIC AVE.
ST. LOUIS
UNION AVE.
BLUFF ST.
FRANKLIN ST.
BELLEVIEW AVE.
BENNSYLVANIA
JEFFERSON
BLUMMIT ST.
GENESSEE ST.
WYOMING
LIBERTY
HICKORY
MULBERRY
K.C.FT.S.& M.R.R.
K.C.SUB.BELT R.R.
CENTRAL
OHIO
WATER
LYONS
CENTRAL AVE.
OHIO AVE.
JAMES
ANN AVE.
ARMSTRONG AVE.
MINNESOTA AVE.
BARNETT
DUGARRO
OHCHARD
C.D.W.Ry.
Flow Line Bridge
James St. Bridge

FIG. 1

Kansas City is located on the south side of the Missouri River, below one of its great bends, and lies in the States of Missouri and Kansas, with the crooked Kaw River flowing through it to join the greater stream. While commercially and socially possessed of common interests, the community has political and governmental divisions, due to its position on the border line of two States, and is divided into two distinct cities with their various suburban settlements.

The valley of the Kaw River, popularly called the "West Bottoms," is about a mile wide and is quite flat; it is skirted on each side by high bluffs and hills, while the channel of the river meanders through it, in its lower reaches lying close to the western bluff. The Missouri River has been gradually pushed northward, through the agency of dikes and mattresses, and has deposited, in the last fifteen years, a strip of land, about one-quarter of a mile wide, extending from side to side of the valley of its tributary. While practically the entire residence portions and the principal retail business districts of both cities are on the hills, there are to be found, crowded into the Kaw Valley, many commercial industries, packing houses, stock yards, manufacturing plants, wholesale warehouses, the freight stations of nearly all the thirty-four railroads entering the city, several large terminal freight yards, and the present union passenger station. As an essential adjunct, there is, throughout the bottoms, a complicated network of railway tracks, occupying streets and alleys as well as private rights of way.

The transaction of the business of the cities involves an enormous traffic into this valley from both sides, and all traffic from city to city must pass through it. A very large portion of the freight consigned for local consumption must be hauled out of the bottoms; some up-town distributing houses haul goods out, store them, and then haul them back; the packing houses, factories, and other industries are continually conveying their products to both cities; and, too, a great deal of the merchandise consumed in Kansas City, Kans., is hauled from Kansas City, Mo.

All this traffic is confronted by very inadequate passageways and very heavy grades from valley to hills, not less than 5% on the east side, and not less than 6% on the west side. Furthermore, on the east side there are from the northern part of the city only two possible entrances to the bottoms; and, on the west side, giving access to

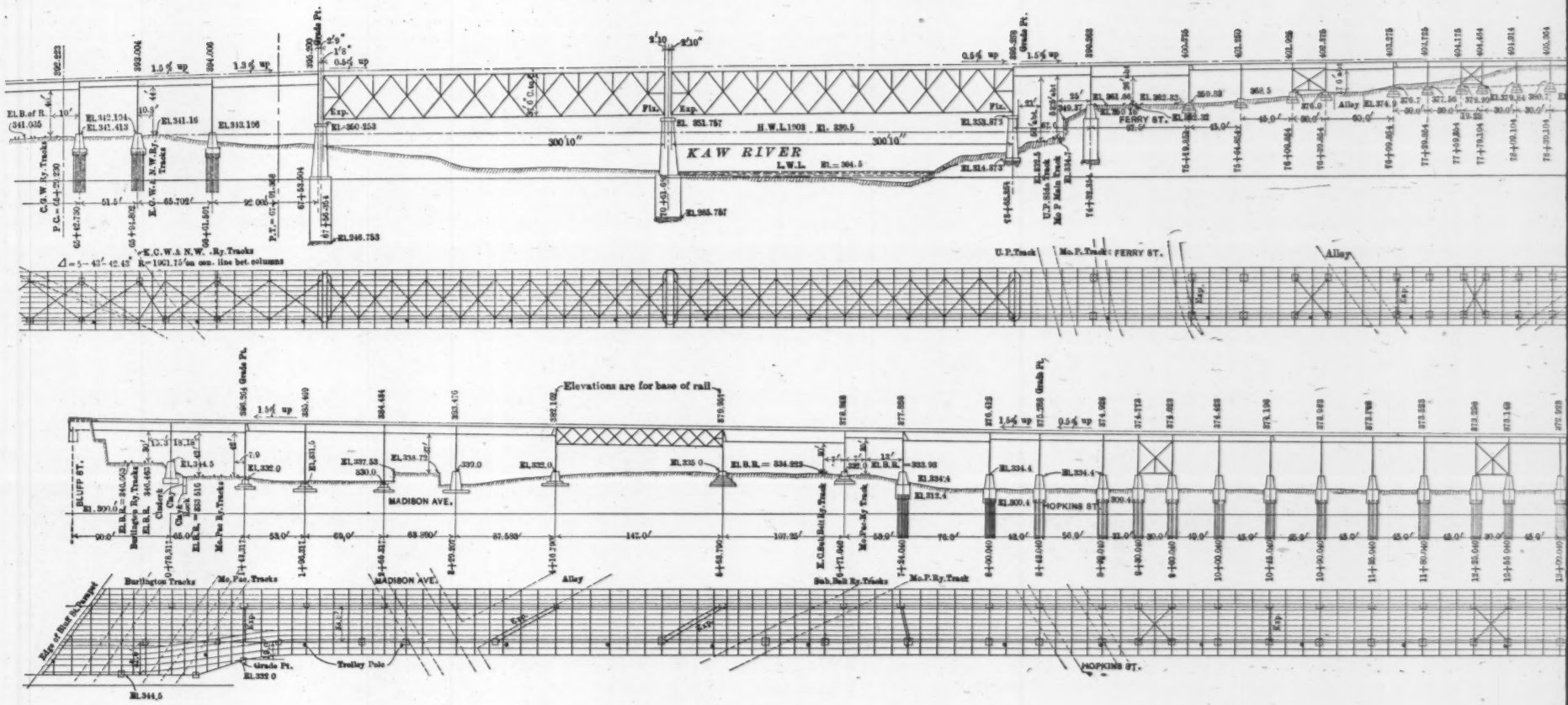
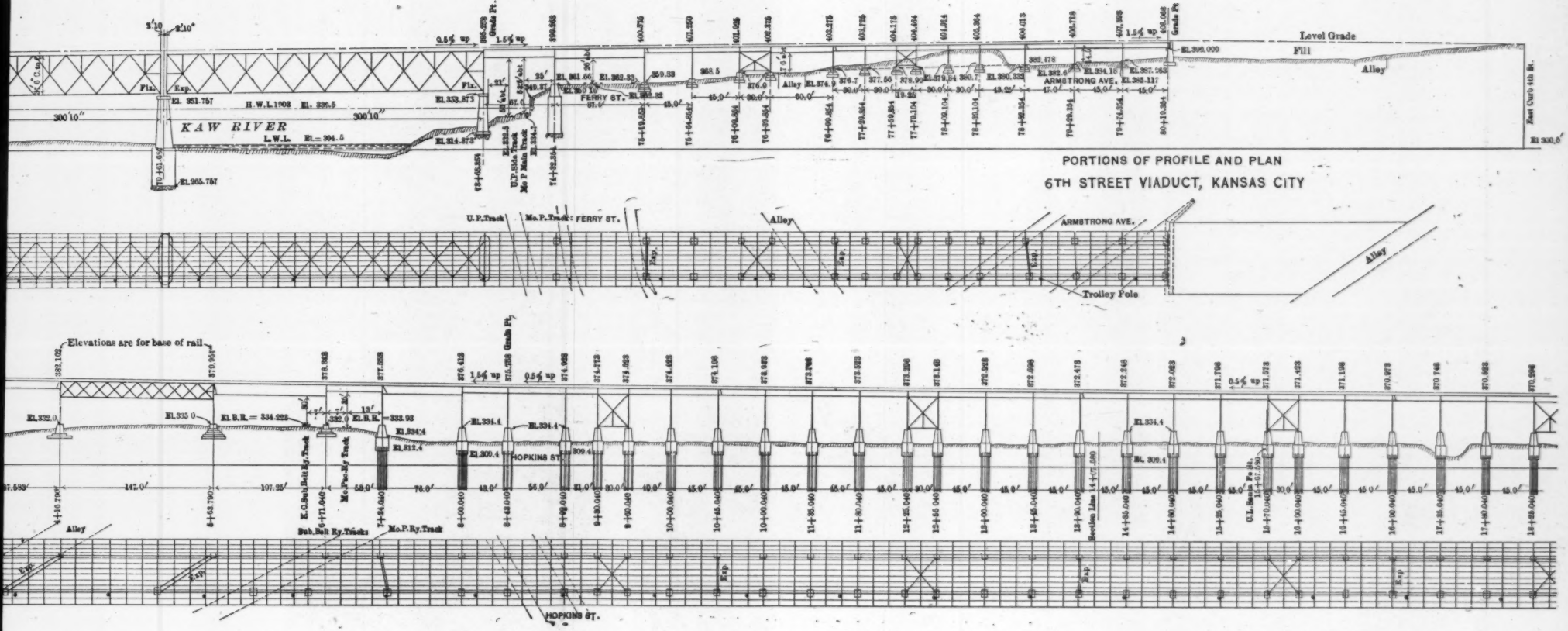

KAW RIVER
FERRY ST.
MADISON AVE.
HOPKINS ST.
BLUFF ST.
Elevations are for base of rail
Burlington Tracks
Mo. Pac. Tracks
Alley
Trolley Pole
U.P. Track
Mo. P. Track
K.C.W. & N.W. Ry. Tracks
Sub. Belt Ry. Tracks

PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1120.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY



PORTIONS OF PROFILE AND PLAN
6TH STREET VIADUCT, KANSAS CITY

Minnesota Ave., the principal business street of Kansas City, Kans., there is only one passageway. In addition to the steep grades to be contended with, the vehicular route to every freight station is over dangerous grade crossings frequently congested with traffic, while the interurban travel is compelled to follow circuitous routes.

Not only does the highway traffic to and from the bottoms operate under adverse conditions, but the street railway facilities are inadequate, there being only two through lines to handle the entire between-city travel. One of these is a very circuitous surface line, operated at great discomfort because of the many grade crossings, the steep grades, and the congestion of streets in the bottoms. The other line is elevated for a considerable part of the way, but the route is indirect. The need of additional rapid transit across the bottoms was long keenly felt by the street railway interests and the public, especially the latter.

Various measures have been devised from time to time to expedite this travel, and a viaduct has often been advocated. Shortly after the disastrous flood of 1903, which, by the inundation of the entire west bottoms, for a time completely cut off all intercommunication, a plan was proposed for the two cities to build jointly a street-car and vehicular viaduct, but it soon became apparent that no money was or would be available for such a purpose, and in any case the difficulties of two city governments jointly handling a structure located in two cities, two counties, and two States, seemed almost insuperable.

The proposition to build a toll viaduct by private capital for an investment was then broached, and investigation was undertaken to determine the feasibility of such a plan. A preliminary survey showed that a viaduct could be built across the accreted lands near the Missouri River, then unoccupied, which would shorten the distance from center to center of business districts by street car 4 150 ft. (26%) and 18 min. (60%); would give easy grades and uninterrupted passageway from city to city for highway travel; and, by a judicious location of laterals and approaches, could give convenient access, with elimination of all grade crossings, to each of the various freight stations in the bottoms.

Careful investigation, extending over a number of months, was then undertaken to determine: First, the amount of time to be saved by the patrons of a viaduct; second, the deterioration of equipment to be lessened; third, the additional loads to be carried per vehicle;

fourth, the actual money to be saved by the user; fifth, the amount of toll the user could afford to pay and still profit by the use of a viaduct; sixth, the probable volume of traffic to patronize such a structure; seventh, the effect of the viaduct on the distribution of population, and the consequent increase of travel; eighth, the probable growth of the cities and the increase of business; ninth, a minute and thorough estimate of the cost of construction; tenth, the annual cost of operation; eleventh, the annual income; and twelfth, the earnings to be realized.

For many of these questions there was no precedent, and some conclusions were based on unsatisfactory data. The determination of the number of vehicles passing to and from the bottoms required merely an extended series of actual counts, secured from day to day, principally by detectives of the Pinkerton service. These counts were made at various times from October, 1903, to June, 1904; and numerous independent tallies agreed closely with a total of about 12 000 vehicles per day to and from the bottoms.

As a further verification of these figures, statements were obtained from leading transfer men and large haulers as to the estimated use they would make of the viaduct, the total sum of which showed an entirely satisfactory figure.

A study of comparative statistics showed Kansas City to have exceptional business prospects, for, while at present twenty-second in population, it ranks as ninth in bank clearings, twelfth in total postal business, second in railroads, second in grain business, and it is situated in the center of a rich and productive territory.

Neither did the estimates of cost of construction and maintenance offer particular difficulty; but the determinations of the individual profit to be secured and the toll which could be reasonably expected were but approximations.

The matter of the utilization of the viaduct for street cars was taken up early with the company operating the cars in the city, and with certain suburban lines, with the result that tentative agreements, later duly confirmed by contracts, were made for traffic arrangements.

The conclusion of the preliminary investigation was that the structure as then planned, reaching from city to city, with laterals at properly selected points giving uninterrupted access to the freight houses,

was feasible as an investment, could be built for about $2 275 000, could be operated for about $180 000 per annum, and could earn from 5% upward on the capital stock, depending on the future growth of the city.

The Kansas City Viaduct and Terminal Railway Company was organized to build and operate such a viaduct, and the Common Councils of the two cities granted to the Company the necessary franchises. The principal provisions of these franchises are:

The franchise is to continue in force for thirty years;

A schedule of maximum tolls is fixed;

A clause provides that the City may at the end of ten years purchase the viaduct at any time, by notifying the Company one year in advance, at the actual cost and a stipulated varying interest rate;

A provision is made for the Company to pay the City 2% of its gross revenue.

As soon as certain necessary legislation was obtained from the Missouri Legislature, financial arrangements were completed, right of way was secured, and the construction, for which general contracts had already been made, was begun.

It had been the intention to commence the Mulberry Street Approach first, to finish that approach and the east end of the main structure from the approach to the eastern terminus, and have this portion in operation before the completion of the main viaduct; but the Company's attorneys discovered many difficulties in getting right of way for the approach, so the construction of the Main Viaduct was proceeded with until the legal complications could be untangled.

The following pages contain a discussion of the design and construction in the following order:

1.—Superstructure, design, manufacture, and inspection.
2.—Substructure, design, and construction.
3.—Engineering field methods.
4.—Erection of superstructure metal.
5.—Floors, concrete floor, paving, street-car deck, hand-rail, and lighting system.
6.—Approaches and laterals.
7.—Final costs.

1.—Superstructure.

The design provides a roadway 30 ft. wide over a part, and 38 ft. wide over the remainder of the structure, to carry a loading varying according to span from 50 to 100 lb. per sq. ft., or a 15-ton road roller; and, separated from the roadway by a substantial hand-rail, a double track for electric cars, to support a continuous line of cars on each track, each car weighing 40 tons and having a length of 43 ft. 3 in. over all. Each of these assumed loads is increased by an impact percentage as noted hereafter.

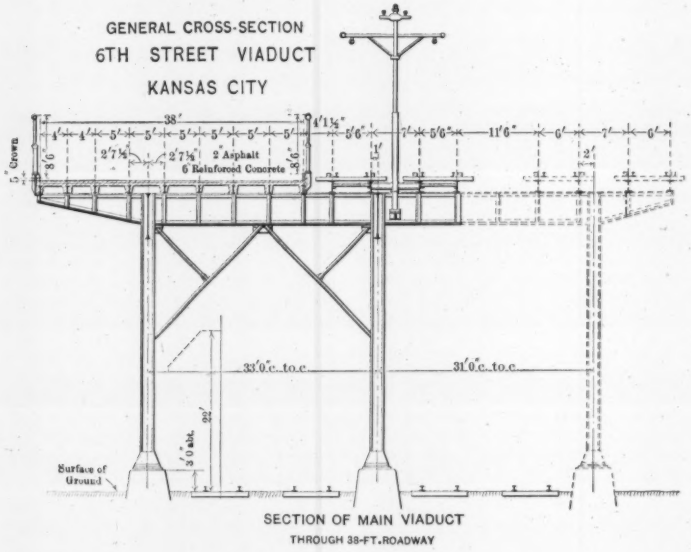


Fig. 2.

Before adopting the layout for the spans, an extended economic study was made by preparing estimates for different arrangements of girders and columns. The varying amount of shopwork, and the unit cost of material, as well as the total weight of metal per linear foot, were duly considered, the resultant variation of substructure cost with span length was estimated, and the most economical girder length, floor arrangement, and tower spacing for the confronting conditions were definitely determined.

This led to the selection of a two-column structure with one column

about under the center of the roadway, and the other about under the center of the motorway, with longitudinal girders riveted into the columns, cross-beams riveted into the girders and columns, and stringers laid on top of the cross-beams.

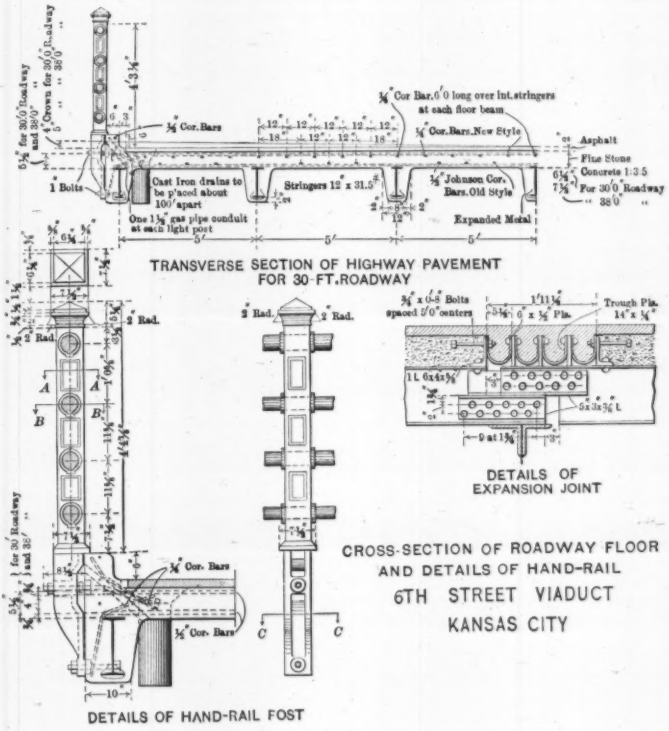


FIG. 3.

Considerations of possible increase of railway traffic resulted in the decision to arrange the viaduct so that at any time there could be added two additional tracks, each to carry a continuous line of cars, each car weighing 70 tons and having a length of 43 ft. over all. This is to be done by adding a third row of columns with girders, and splicing the cross-beams to the original motorway cantilever beams.

The two columns are 33 ft. from center to center transversely, and spaced so as to be loaded equally when the final loading is applied.

The third column will be 31 ft. from the original motorway column and at mid-point between the two added tracks. The transverse bracing is arranged so that tracks may be laid on the ground between the columns, and occasional turnouts are allowed for by spans of special length.

Longitudinally, the structure is divided into sections composed of one span of 30 ft., and seven spans of 45 ft. The 30-ft. spans have longitudinal bracing to care for the entire thrust of the 345-ft. section, thus imposing very few obstructions to free transverse passageway below, a stipulation of certain river-front property holders. The mid-girder between the towers is supported at one end in a pocket or shelf so as to slide and allow for expansion.

The highway floor is of concrete, enveloping the steel stringers, and supporting a pavement of asphalt. Hand-rails are placed on each side of the highway. The car tracks are of ordinary rails with the usual creosoted pine ties and guard-rails. The maximum grade on the main viaduct is 1.5%, and the minimum is 0.5%, this latter being maintained so as to drain the asphalt pavement properly.

The following working intensities were used in the design:

Tension Stresses.

	Pounds per square inch.
Flanges of plate girders (counting in one-eighth of the web)	14 000
Rolled shapes	16 000

Bending Stresses.

Extreme fiber of rolled sections	16 000
Extreme fiber of timber beams	2 000

Compression Stresses.

Top chords	$18\,000 - 70\,\frac{l}{r}$
Inclined end posts	$18\,000 - 80\,\frac{l}{r}$
Intermediate posts and diagonals	$16\,000 - 80\,\frac{l}{r}$
Lateral struts and bracing	$16\,000 - 80\,\frac{l}{r}$
Columns	$16\,000 - 60\,\frac{l}{r}$

Shearing Stresses.	Pounds per square inch.
Webs of plate girders	10 000
Rivets	10 000
Impact, for motorway loading	$I = \frac{400}{L + 500}$
Impact, for highway loading	$I = \frac{400}{L + 150}$

Under extreme temperatures and full live load, the shortest columns were computed to have an extreme fiber stress of 24 000 lb. per sq. in., but this was not considered excessive for such unusual conditions.

All calculations were based on the use of equivalent uniform live loads instead of actual wheel concentrations. Because of the very considerable labor involved in computing anew, for each slight change in panel length, the cross-beam reactions, the results from a series of panels of different lengths were plotted and a curve was drawn connecting all points. This diagram proved of great service in designing the many special girders, for a principal part of the calculations was saved by taking off directly the amount of the imposed loads.

Trestle Construction.

The various special girders and columns were detailed in conformity with the standard portions of the structure, and may be generally included in the following description.

Stringers.—Both the highway and the motorway stringers consist of I-beams set on top of the cross-girders and cantilevers, and connected thereto by rivets or bolts. Small shims are used under the highway stringers to provide a crown for the roadway. All stringers are continuous for the corresponding girder length, either 30 ft. or 45 ft., and are joined by small splice-plates. At expansion joints, stringers are halved and reinforced with bearing angles so that they slide upon one another. Connecting the motorway stringers, there is a rigid system of bracing, composed of single-angle diagonals, and of cross-struts of either one angle or one channel. The connections for all this bracing are made by lug-angles to the webs of the beams so as to leave their upper flanges clear for convenience in laying the ties.

Cross-Girders and Cantilevers.—Cross-girders are spaced at 15-ft. centers, thus placing two intermediate beams in the 45-ft. girder spans and one in the 30-ft. spans. They are riveted to the webs of the main girders or to the columns, with their upper flanges level with the tops

of the main girders. These beams are 52 in. deep, with ⅜-in. webs, and flanges composed of two angles. The intermediate stiffeners, which are pairs of angles placed at each concentration point, are crimped, while the end stiffeners have fillers on the web. The cantilevers at each end of each cross-beam follow the same detail, that on the highway side varying in depth from 52 to 12 in., though the motorway cantilever is of full depth to its end, where provision is made for splicing on a future cross-beam. Splice-plates extend over the girders or columns, and connect the top flanges of the cross-beams and cantilevers. At columns, the thrust at the lower flange is carried into the cross-beams with cast-iron thrust-blocks bearing against the channels of the posts and the webs of the main girders. These blocks were cast to a driving fit, were driven to place after the field riveting was completed, and were also supported by bolts through the column webs.

Main Girders.—The standard 45-ft. girders are 72 in. deep, with ⅜-in. webs and duplicate flanges of two angles and cover-plates. The general details are the same as for the cross-beams, except that the intermediate stiffeners are spaced 5 ft. apart.

Columns.—The columns, throughout, are of two rolled or built channels joined by a built I, with the webs of the channels in general parallel to the longitudinal axis of the structure. Stay-plates connect the flanges of the channels. Columns extend clear to the top of the main girders and cross-beams, providing for them full riveted connections. Lug-angles and brackets are riveted to the columns for the connection of the bracing, and shelf-angles are provided for convenience in girder erection. The shoe is built of side-plates and angles, fastened to the channels, and distributing the load to two heavy base-plates. Curved plates, 18 in. high, are fastened to each side, as sleeves for the anchor-bolts, and as bearings for the anchor-nuts.

At expansion points the columns have on one side pockets made of plates riveted to the channels supporting a bottom or base on which the girders slide. The tops of the columns are tied back to the adjacent span in a manner to relieve the main connection rivets of tension due to eccentric loading.

Bracing.—Lateral bracing is used only on curves, and in each tower span, it being considered that elsewhere the concrete floor would give all needful lateral rigidity. Where used, it consists of two diagonals of two angles each, with sufficient area to take the stress in either tension or compression. These angles are placed just below

FIG. 1.—THE COMPLETED VIADUCT, LOOKING FROM THE EASTERN END TOWARD KANSAS.

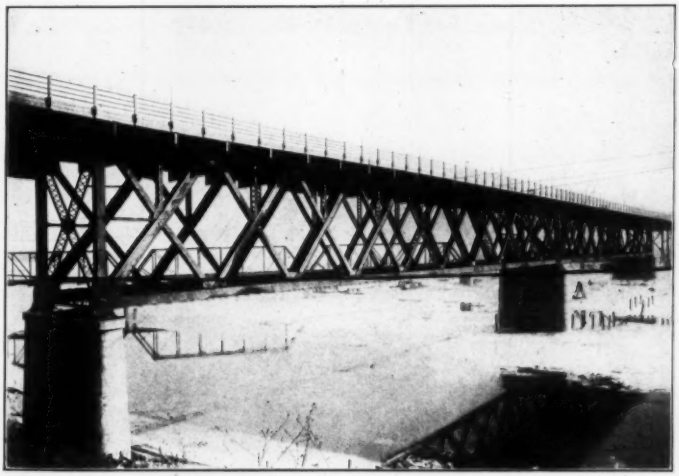


FIG. 2.—THE COMPLETED BRIDGE OVER THE KAW RIVER.

the cross-beams, are riveted to the column beams and to the main girders, and also to each other beam they touch.

Longitudinal bracing at the towers is made up of two diagonals and a lower horizontal strut, all composed of four angles, laced, I-shaped, and of dimensions to fit into the columns. At the upper points the diagonals are attached also to the longitudinal girder.

Transverse stiffness is provided by knee-braces of two angles fastened to the column webs, and to the lower flanges of cross-girders, each column being braced in this way. The ends of pairs of girders resting in the expansion pockets are connected by a light bracing frame.

Truss Spans.

At the Kaw River are used two truss spans, each 300 ft. 10½ in. from center to center of end pins. They are of the double-intersection, deck, Warren type, with subdivided panels, there being ten main panels. The depth from center to center of chords is 37 ft. 6 in., and the trusses are spaced 33 ft. apart. These spans are riveted throughout, and attention is called to their exceptional size; each span weighs about 900 tons.

The floor system is arranged just as on the trestle, with cross-beams and cantilevers riveted into the top chords and short vertical posts. Two small expansion joints for the floor are used on each span to avoid the distortion of the floor-beams as the top chord shortens.

The top chords are of two web-plates, a cover-plate, and four angles, with the upper angles placed inside. The various top chords are spliced only enough to assure alignment, and the contact of abutting webs is relied on to carry the compression. The bottom chords are of two built channels laced with angles. The mid-span chord has an area of 212 sq. in.

Diagonals are alternately of built channels turned out, and of built I-struts, the latter passing through the former. Thus the channels rivet on the outside of the gusset-plates and the I's between them.

The upper laterals are of two-angle diagonals placed in the plane of the lower edge of the chord. They reach the chords only at main panel points, and therefore intersect at the intermediate floor-beams, to which they are attached by plates passing through slotted holes in the webs, and are connected thereto by lug-angles. The lower laterals consist of diagonals, between main panel points, of laced, four-angle, I-struts, the depth of the chords.

The upper laterals are made heavy enough to dispense with inter-

mediate transverse bracing between the trusses. Between the end posts there is cross-bracing of four-angle I-struts from the floor-beam above to a transverse strut below. This arrangement avoids the transference of loads from truss to truss due to unequal deflections, and also the unsatisfactory details of transverse bracing fastened to other than vertical members.

The shoes are built up of plates and angles, and are connected to the trusses by 12-in. pins. Ten segmental rollers, 8 in. high, support the expansion shoes, and cast-steel base-plates are used at each end.

The trusses were calculated on the assumption of all loads being equally distributed between the two systems. Although the trusses are equally loaded when fully loaded, the resultant stresses vary somewhat, because the larger part of the load on the highway truss is from the concrete floor, while on the motorway truss the live load is much greater. Both were calculated, and, in the make-up of sections, the chord stresses of the highway truss were combined with the web stresses of the railway truss and the two trusses were made identical.

The one other truss bridge of the viaduct is a skewed span, 147 ft. long, over railway tracks. This span is of the same general type as that at the Kaw River, except that the intermediate vertical posts are absent, the floor-beams being supported on the gusset-plates. The depth of trusses is 15 ft. 6 in. At each end, owing to the skew, there is a diagonal girder, 62 ft. long, and the regular cross-beams are riveted into it. The span is supported on shoes of the usual type, that at the roller end resting on seven segmental rollers supported on a cast-steel base. There is no bracing, either longitudinal or transverse, in the columns under this span, and especially heavy sections were used. The columns supporting the fixed end are 42 by 32 in., having about 200 sq. in. of section. Each of the four columns is anchored to its concrete pedestal by eight 2¼-in. bolts 9 ft. long.

Specifications and Manufacture.—The consulting engineers prepared complete detailed drawings, and from these the contractor prepared working drawings. These engineers' drawings do not give in full all minor details, such as the exact spacing of rivets, sizes of small plates, etc., but do fix the dimensions of all members and of all parts thereof, allowing the shops to fill in the lacking dimensions according to their preference. The shop drawings were then checked by the consulting engineers, who made the necessary corrections, and finally approved all details. As a representative, located at the shops,

gave all points prompt attention and decision, much time was saved and correspondence avoided.

All metal is open-hearth, medium steel. The specifications provided that all plates should be rolled from slabs of at least six times the plate thickness, which slabs were to be made by rolling an ingot and cutting off the scrap. The following maximum percentages of ingredients were fixed:

Phosphorus, 0.04 to 0.06; sulphur, 0.04; silicon, 0.04; manganese, 0.70. The ultimate tensile strength on test pieces was to be from 60 000 to 70 000 lb., and the least allowable elastic limit, 35 000 lb. The least allowable elongations on specimens varied from 20 to 24%, with a corresponding reduction of area of from 44 to 36 per cent.

First-class workmanship in every respect was specified. Sheared or hot-cut edges of plates were planed so as to remove $\frac{1}{4}$ in. of metal. Rivet holes were sub-punched $\frac{1}{8}$ in. less, and reamed to a diameter $\frac{1}{16}$ in. greater than that of the rivet, and it was required that the reamed holes be perpendicular to the metal surface. A modification in part of the foregoing was made, and the field holes for the stringer bracing and certain other minor parts were punched full size.

The ends of all girders and abutting members were planed to make perfect contact. Stiffeners on girders were ground so as to fit tightly against the flanges. This was strictly enforced in the case of stiffeners carrying concentrated loads, and in certain cases where the stiffeners did not quite touch they were removed and new angles riveted on. Edges of spliced web plates were planed and put in contact for their full length.

It was specified that special attention be given to the cleaning and painting of the metal-work in the shop, where one coat of paint was applied. The metal was cleaned of rust, dirt, and loose scale before painting, and all surfaces of metal in contact were required to be painted before being put together. In a few instances it was necessary to have finished work cut apart in order to do this painting.

A large portion of the work was laid out direct, without the use of templates. Gauge punches were used almost exclusively on pieces which duplicated several times. There was much duplication, of which could be mentioned 220 45-ft. girders, 480 floor-beams, 350 cantilever-beams, practically all stringers, and so forth. Although the columns are all of different lengths, groups were made together requiring only the details near the lower ends to be varied.

The rate of shop work varied considerably, but averaged about 900 tons per month, finished and shipped. The Kaw Bridge spans, comprising some 1 800 tons, were fabricated and shipped in five weeks.

Careful inspection was made of all operations in the shop. Rivets were examined as driven; there was a general supervision of laying out, punching, shearing, assembling, etc.; and the finished piece was checked before loading. The result of this careful inspection was apparent in the very few errors found in the field.

Full records of the progress of each piece were kept, and weekly reports were made giving the status of work of all members and the percentage of the estimated total completed. The weights were all calculated from the drawings, and checked against the scale weights.

2.—Design and Construction of Substructure.

Surveys and borings showed bed-rock to be about 50 ft. below the ground surface across the entire valley except for a few hundred feet from each bluff, where it is very much lower. The material overlying bed-rock was found to vary from soft muck and running sand to clay, firm sand, and gravel.

Because of the uncertain bearing capacity of this accreted land, and of a possibility of scour, piles were used under 276 of the 326 pedestals of the viaduct.

The first design contemplated the use throughout of a patented form of concrete pile, and the pedestals were designed accordingly. This patented pile was to have been built by driving into the ground a steel shell or mould having a removable point, and on reaching the required depth the shell was to be filled with concrete and gradually pulled out as the concrete was tamped into place. Further consideration and examination of driven piles made it seem to be undesirable and inexpedient to use this patented pile in the wet soil to be encountered.

The manifest desirability of a concrete pile made the engineers loath to abandon that type of foundation, and led to the adoption of a built-up driven pile. A number of experimental built-up piles were made at once, and, after the feasibility of driving them was demonstrated, their general construction was immediately undertaken.

Of the pedestals not supported on piles, some rest on rock, some on hard clay, and some on soft material. Thirteen pedestals near the east

end of the viaduct are under the buildings of a machine shop or under railroad tracks. The material here is very soft, and, after experiments, an intensity on the soil of 1 ton per sq. ft. was allowed. In order to have the excavations shallow, the bases were reinforced. Plain steel bars were used.

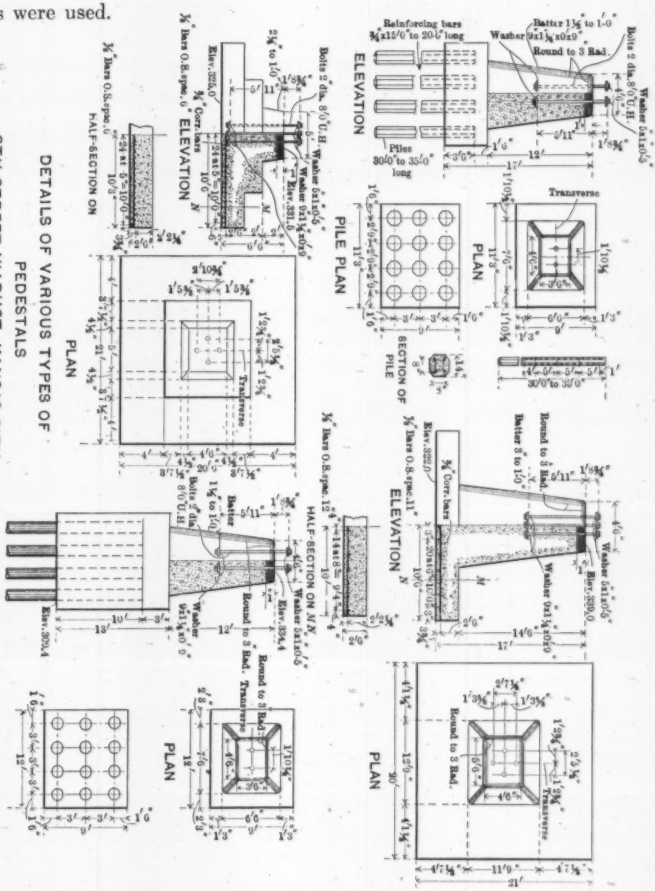


FIG. 4.

Near the western end of the viaduct the pedestals rest on firm red clay, loading it to about 3 tons per sq. ft. The three Kaw River piers are founded on rock, two being sunk by the pneumatic process, and one built by open excavation.

Piles.—The moulded concrete piles are all of one size, 30 ft. long, 10 in. square at the lower, and 16 in. square at the upper end, with a diagonal 3-in. cut off each corner. Each is reinforced by four $\frac{7}{8}$-in. steel bars, extending from end to end, symmetrically placed as near the surface as practicable, and surrounded by a cylinder of heavy woven-wire fencing intended to occupy a position about $1\frac{1}{2}$ in. from the surface of the concrete. In the later piles the wire was omitted.

The forms or moulds for the piles were simple boxes, with bottom, sides, and ends of 2-in. plank suitably cross-braced. Lugs on the cross-braces engaged wooden wedges which tightened and held the parts together yet readily permitted them to be knocked down for moving. The diagonal corner moulding made the form almost water-tight. The upper end of the pile was shaped into a round head to facilitate driving, and the lower end was bluntly pointed. The several parts, of course, were made interchangeable. The interior surfaces were coated with oil before concreting. The piles were moulded lying on the ground, in groups of from 8 to 16, in position convenient to their point of driving.

The specifications provided that the piles be made of concrete composed of "1 part of Portland cement, 3 parts of Kaw River sand, and 5 parts of clean hard broken stone to pass a 1-in. iron ring". The cement used was Iola Portland, and cost, f. o. b. Kansas City, $1.60 per barrel. The sand of the Kaw River is very clean, sharp, and well graded, testing about 52% retained on a No. 30 sieve; its cost, delivered, was 75 cents per cu. yd. The stone was from quarries about the city, and cost $1.30 per cu. yd. delivered. There was considerable trouble in securing clean stone, for the limestone strata of this region are overlaid with clay, and, especially in a rainy season, it is impossible to prevent some clay from intermingling with the stone in the crusher bins.

Some limited experiments were undertaken to determine whether the amount of clay present had a deleterious effect on the concrete. Test bars made of measured quantities of material, mixed in a uniform manner, were prepared with the stone as it was delivered from the crusher, and with stone carefully and thoroughly washed and cleaned. When these bars were tested and compared it was found in all cases that the unwashed clayey stone made stronger concrete than the clean washed stone.

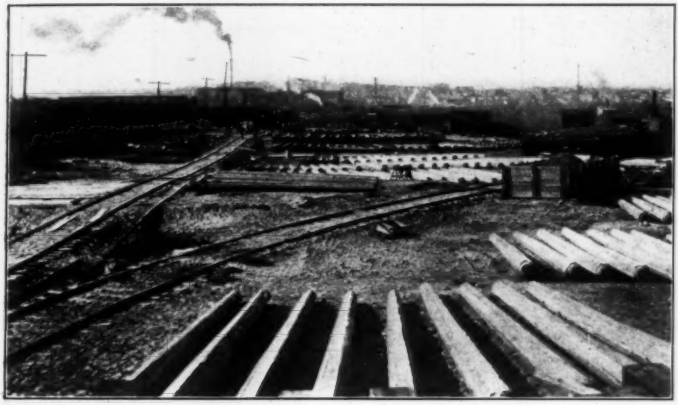


FIG. 1.—CONCRETE PILES: FINISHED PILES CURING.

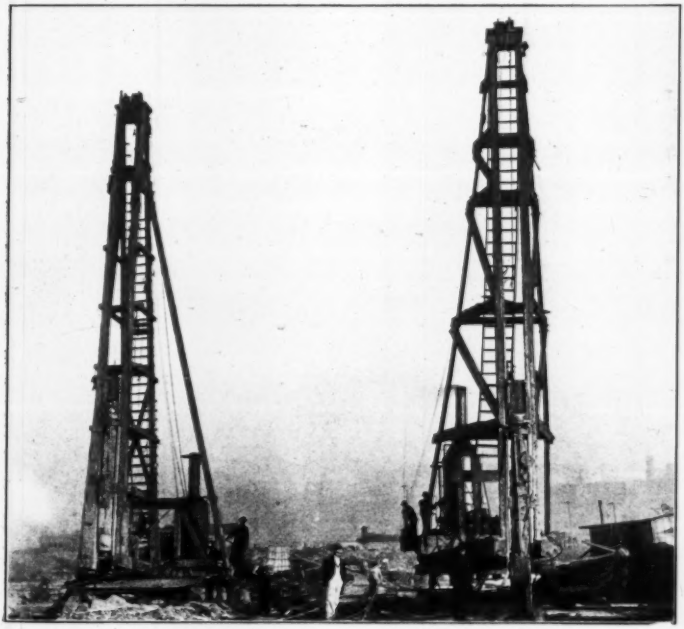


FIG. 2.—PILE-DRIVERS, SHOWING SPECIAL ARRANGEMENT OF FOLLOWERS.

The concrete for the piles was made by hand on tight movable platforms. The practice was to mix together the stone, sand, and cement dry, and then add enough water to give a reasonably wet mixture. The material was distributed by wheel-barrows, and dumped into the prepared forms. After a 2-in. layer of concrete had been spread in the form, the reinforcing of wire and rods, already prepared, was laid in position, the remainder of the concrete was dumped in, tamped to place, and smoothed on top to a reasonably even surface. Sticks were used to tamp through the meshes of the wire and to hold the rods apart. The rods had a tendency to bunch together, and it required constant care and watchfulness to get them where they belonged. Several methods were tried to obtain the desired results easily, but none was entirely satisfactory. The conclusion is, however, that stiff iron spacers, either as a ring or as an *X*, at intervals of about 3 ft. along the rods, would give superior results.

Each pile was marked with the date made, and for several days thereafter was showered copiously with water. The side and end forms were usually removed in 48 hours, but the pile was not rolled from its base for about a week.

Over-anxiety to re-use the base forms without delay caused the loss of several piles, for the heavy green piles were easily cracked by very small inequalities of the ground. Weather conditions, of course, affected the rapidity with which the piles hardened, but they were generally allowed to cure from 3 to 5 weeks before use, although some were driven 2 weeks after being made. Each pile contains 1.4 cu. yd. of concrete. The finished piles are shown in Fig. 1, Plate III.

For driving, or rather sinking, the piles, heavy turn-table drivers, equipped with a special kind of follower, were used. This follower was of timber, about 12 ft. long, with guides to fit the leads at its upper end and mid-point, and with a deep cylindrical collar at its lower end. A standard 3 000-lb. hammer rested on the follower and was fastened loosely to it. The usual "pile line" was carried down, through a steel sheave, then up and made fast to the top of the leads. To this sheave was fastened a 10-ft. length of steel cable having a slip-noose at its lower end, and a "pull-off line" attached to the slip-loop of the noose. A water-jet pipe, consisting of a $2\frac{1}{2}$-in. pipe 35 ft. long, with its lower end drawn down to a nozzle, completed the equipment. This jet pipe was handled by a line to a spool of the engine, and was supplied with

water at a pressure of about 120 lb. through a 3-in. standard fire-hose. To provide the water for the jets, a pumping plant, consisting of a high-duty 18 by 10 by 12-in. fire pump with suitable boiler equipment, was erected on the bank of the Missouri River about $\frac{1}{8}$ mile from the structure, and a pipe line, from 3 to 4 in. in diameter, was laid from it along the entire viaduct. Fig. 2, Plate III, shows the pile-drivers and the special arrangements of the followers.

In general, piles were driven before any excavating was done, and they were sunk, so that their upper ends were from 1 to 5 ft. below the ground surface, in the following manner:

After the driver had been set to the correct position by the adjustment of the rollers on which it stood, the hammer and follower were lifted to the top of the leads. The jet was then played on the desired position of the pile, and, as the ground softened, it was gradually pushed down full length. In the meantime the slip-noose cable was placed around a pile some 4 ft. from its upper end and the pile was raised and swung into the leads where it was straightened up and the follower settled carefully on the protruding head. By means of the sheave-line, the pile was then churned up and down, the pile, the timber follower, and the hammer resting thereon all going up and down together, while the jet was kept in constant service. Drops of as much as 6 or 7 ft. were made, without apparent injury to the pile, and the churning process was continued until the required depth was secured. By the "pull-off" cable, the slip-noose was easily loosened and removed.

The effect of running the jet down first was to cause the pile to keep its correct position, as it was found that otherwise each pile had a tendency to travel toward the adjacent ones previously driven, where the ground was still soft; and even then the pile tended to lean toward the side where the jet was played during sinking. It is believed that more accurate sinking could have been done with a pair of jets, one on each side. The simple slip-noose for handling the piles was developed only after the failure of several more or less elaborate schemes devised for the same purpose.

The time consumed in the actual sinking of a pile varied somewhat, but, where no particular obstructions were encountered, from 2 to 5 min. sufficed. However, the highest record for one gang in 10 hours was 36 piles.

It was soon apparent that, owing to the delay in starting, cold

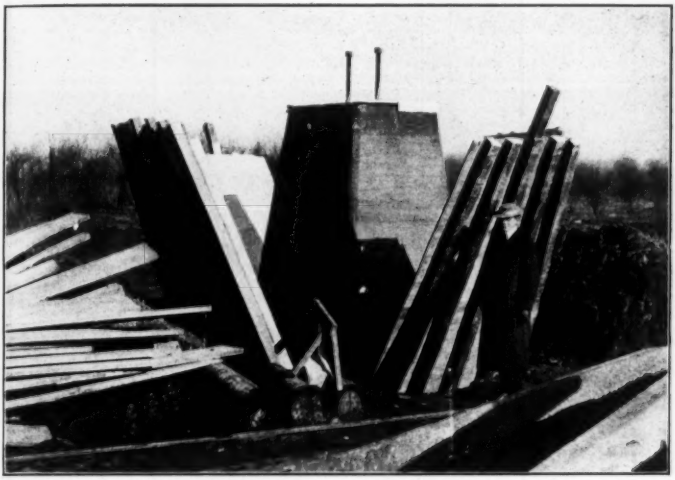


FIG. 1.—PEDESTALS: FORM REMOVED.

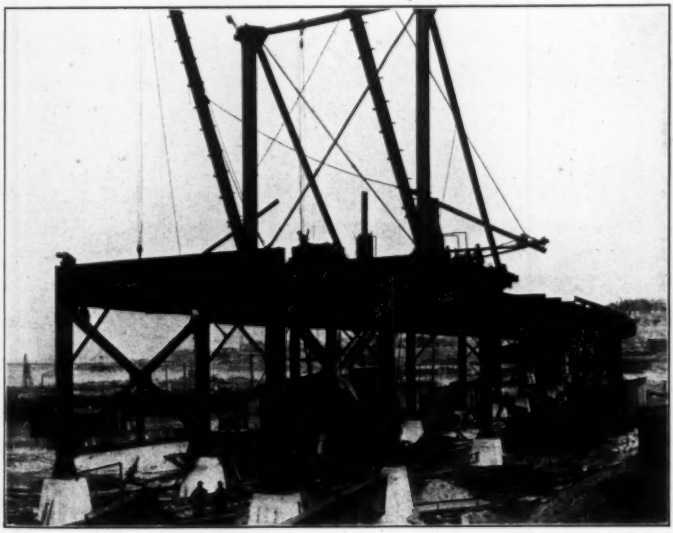


FIG. 2.—TOWER SPAN, WITH LONGITUDINAL BRACING.

weather would probably set in and stop concreting before all the concrete piles could be made, and it was necessary to devise a substitute so that the structure might be finished in the time assigned. Creosoted pine piles were, with reluctancy, decided upon; a rush order was given to pile contractors and creosoting plants, and, by constant attention, the piles were rapidly delivered.

The specifications called for:

"Piles cut from live, straight, long-leaf, yellow-pine timber; to be free from cracks or wind-shakes; to be so straight as to be at no point more than one-third of their sectional diameter out of a straight line joining the centers of the ends; to be of even, gradual taper from end to end; and to be not less than 10 in. in diameter at the small end."

The specifications for creosoting provided, after the usual requirements for dressing, peeling, and artificial seasoning, that:

"After the seasoning is entirely finished, pure, unadulterated dead oil of coal-tar shall be injected into the timber so that to each cubic foot of timber will be 12 lb. of creosote oil."

Additional equipment for handling the timber piles was hastily provided. The drivers were of the ordinary light timber-framed type, braced by guy lines to the top of the leads, which were 60 ft. high. Each was mounted on two wooden rollers, 1 ft. in diameter and 20 ft. long, resting on timber sills laid on the ground. Lateral movement was secured by sliding the driver along the rollers. This roller arrangement proved good, permitting rapid moving and setting. Both drop-hammers and steam-hammers were used, the latter doing much more efficient work. Water-jets, arranged about as previously described, were used. The efficiency of the jets was demonstrated in several instances when, because of a break-down of the pumps, attempts were made to drive without water, for these invariably resulted in the piles being injured and broomed, before they had reached their full depth, thus precluding further driving.

On a portion of the structure, 50-ft. piles were driven, reaching to bed-rock; on the remainder, 35-ft. piles were used, resting, as do the concrete piles, on gravel and on clay.

To determine the bearing capacity of a concrete pile, a test load was placed on one of the first piles sunk. The pile loaded was of standard size, was made on August 28th, driven on September 15th in 25 min., and the loading was begun on September 24th. To support

the load, a timber platform, 12 by 4 ft., was balanced on a single post resting on the pile head. Observations were made with an engineer's level, reading on a scale fastened to a piece of 1-in. pipe which was set into the pile a few inches and projected above the platform without being in contact with it. The load was of pig iron, each pig weighing about 66 lb., and was placed by hand at an average rate of 6 tons per hour.

During the first 1¾ hours the load increased from 4 tons (the weight of the platform) to 19 tons, with a very small settlement. This load was constant for the next 15½ hours, and the pile settled 0.012 ft. During the next 4 hours the load increased from 19 to 35 tons, with an additional settlement of 0.038 ft. After an interval of 1½ hours, loading was resumed for 4 hours and 40 min., with a settlement of 0.097 ft. During the 17 hours and 40 min. this total load was kept on the pile, there was a further settlement of 0.062 ft. In the 4½ hours of unloading, the load on the pile was reduced from 43.75 to 4 tons, and the pile rose 0.021 ft., leaving a net settlement of 0.198 ft.

These results made necessary some revision in the pedestal design, as very much greater capacity had been claimed for the patented pile. Moreover, the material varied so much from place to place that this test was not believed to be representative of the entire structure, and some ready method for tests was desirable. It was clearly impracticable to load each pile to determine its bearing capacity, and as there is no way to estimate the capacity of piles sunk by jet, the following plan was adopted.

Creosoted pine piles, 40 ft. long, were furnished and called "test piles." In cases where, by reason of special softness of the ground in driving adjacent pedestals, or from the data of the borings, there was uncertainty, one or two of these "test piles" were substituted for the concrete piles. These were driven by using the drop-hammer in the ordinary way, without a jet, and before any jetting had been done in the immediate vicinity. Observations were made of the drop of the hammer and the penetration of the pile from 25 to 30 ft. down, or about the position to be occupied by the concrete piles, and the resultant bearing capacity was calculated by the well-known formulas. In all cases these results gave a greater safe load than that actually on the piles, in fact, it was usually necessary to resort to the jet to get the timber piles to the full depth.

In the main viaduct there are 798 concrete piles, a total of 23 950 lin. ft. in place; and 2 573 creosoted timber piles, aggregating 93 800 lin. ft. in place; a total of all piles of 117 750 lin. ft.

Excavation and Concreting.—The excavation for pedestal bases was comparatively simple, and the work was done by hand. In nearly all instances where concrete and the shorter timber piles were used, no sheeting or bracing was needed, the sandy, clayey material standing to depths of 8 or 9 ft. For about 80 pedestals, most of which required pits some 13 ft. deep in very boggy, sticky ground, sheet-piles of 2-in. plank were driven by hand around timber frames, and these frames were dropped down to brace the planks as the excavation proceeded.

As soon as the piles were sufficiently exposed they were sawn off at the required elevation. Before concreting they were cleaned of mud and filth by scraping with a hoe having a curved blade. Where concrete piles were not settled to full depth they were cut off with chisels, and the reinforcing bars bent over.

A number of pedestals, particularly those at the east end, under buildings and near tracks, required special attention in excavation with regard to bracing and shoring. One building, under which several were constructed, having been through two fires and a flood, was in a precarious condition, especially as it rested on soft flowing material. The base of one pedestal under it extends under the bed of an engine which was kept in operation by supporting it temporarily until the concreting was completed. For certain railway tracks carrying a heavy and frequent traffic, it was necessary to drive temporary trestles to permit the construction of adjacent pedestals.

Forms.—In most cases no forms were used for the bases of pedestals, the pits being merely filled with concrete to the required elevation. The pedestal shafts are practically of uniform dimensions, and the forms were devised for repeated use. Each side was built as a single piece, of 1-in. horizontal, matched, dressed boards, with vertical posts of 6-in. square timbers. The four sides were held together by three square frames or hoops of 3-in. plank, bolted at each corner, bearing sidewise against the studs. Wedges were driven behind all posts not properly bearing. These shaft forms were easily adjusted to correct position by the insertion of small wedges under the bottom. Sheet-iron corner moulds were used to give the concrete a curved corner of 3 in. radius. For the vertical edges, probably the best results were secured

by using sheet iron of No. 12 or 14 gauge, rolled to approximate radius and then nailed firmly to one side of the form and tacked to the adjacent side only enough to keep the concrete from getting behind. No particular harm was done to this corner mould in pulling down the form. The upper corners were shaped by sheet-iron or wooden moulds, or by the use of a curved trowel. The latter, in the hands of a skilled workman, gave excellent results, but its use was deprecated because of apparent desire to use unskilled labor. Two wooden templates for supporting and holding in place the anchor-bolts, all of which were placed as the concreting proceeded, were fastened to the form.

Concrete.—The concrete was specified to be mixed by machinery in the proportions: One part of Portland cement, three parts of clean, coarse, sharp sand, and five parts of hard, clean, broken stone to pass a 2½-in. iron ring; the proportions to be determined by volume; all ingredients to be measured loose; and enough water to be used to form a wet concrete.

In addition to the usual requirements for package delivery, etc., the specifications for the cement were: To be so fine that 97% in weight will pass a No. 74 sieve, and 90% pass a No. 100 sieve; to be of such tensile strength that briquettes of neat cement will show, for 1 day, from 150 to 250 lb. per sq. in.; for 28 days, from 400 to 600 lb. per sq. in.; to show no drop in strength; to be slow-setting; preliminary set in not less than 30 min., final set in not less than 3 hours; to undergo boiling and steaming tests without failure.

Sand was defined as "particles of hard, clean stone which will pass a ¼-in. mesh sieve, and not less than 50% of which shall be retained on a No. 30 sieve." Both the Kaw and Missouri Rivers have great supplies of excellent sand, that of the former being somewhat cleaner, but both coming within the requirements.

"Sound, hard rock, free from all dust and dirt," was called for, but, as has been noted, the output of local crushers, containing more or less clay, was used. It is difficult to state just how much clay was allowed to be in the stone, as a given load would seem very muddy when wet, yet clean when dry. The stone was generally accepted when there were no lumps of clay, or of clay and stone chips, even though many of the stones were muddy. Inspectors were stationed at the quarries to examine the material as loaded, yet, in spite of all care, some stone was used which contained small balls of clay.

Special detailed specifications in regard to mixing concrete were distributed among the various sub-contractors, to assure uniformity of product.

The concrete was mixed so wet that no real "tamping" could be done, but it was deposited approximately in layers of from 9 to 15 in., and "thoroughly spaded" by thrusting a spade to the full depth of the blade at intervals over the entire area, special attention being given to spading near the surfaces of piles and forms, so that the mortar would have perfect contact. It was in general handled from the mixers in wheel-barrows, either on the ground or on elevated runs, as conditions required, and was thrown freely from the tops of the pits or forms. For other pedestals and for certain abutments, slightly more equipment was needed, and derricks with boxes of simple well-known types were used in handling materials. Anchor-bolts were usually placed after the lower half of the shaft of the pedestal had set, and they were very carefully adjusted to exact position. The upper foot of each pedestal was of rich concrete made with Joplin flint or granite chips. One of the pedestals, with the form removed, is shown in Fig. 1, Plate IV. The pedestals were finished from $\frac{1}{2}$ to $\frac{3}{4}$ in. low, to allow for grouting under the shoes.

Various types of mechanical concrete mixers were used, both batch and continuous machines. The continuous mixers automatically measured the sand, stone, and cement by means of worms, and were mounted so as to be easily portable. Although considerable pedestal concrete was made with these machines, the results were not as satisfactory as those produced by rotary batch mixers which mixed the concrete in revolving drums or boxes. The output of the continuous mixers was not uniform, even though they measured the materials accurately, especially where the vicissitudes of the work necessitated intermittent operation, with frequent stops and starts; the water was not easily regulated, and the mixture was not homogeneous. Moreover, after a time, numerous spots appeared on the surface of the concrete from these machines. It was found that these were caused by small balls of clay, from $\frac{1}{8}$ to $\frac{3}{4}$ in. diameter, embedded in the concrete, the results, of course, of clayey stone. Careful examination, however, proved that these clay balls were practically absent in the work of the batch mixers, showing that the churning and tumbling had been sufficiently vigorous to pulverize and distribute all lumps of clay, with consequent harmless-

ness; while the mixing of the continuous machines had not been sufficiently vigorous to do so and therefore was inferior.

Certain pedestals, abutments, and retaining walls had special forms, made of 2 by 6-in. matched, dressed flooring, laid on vertical studs spaced from 2 to 3½ ft. from center to center. One side of each form was braced to the ground, and the other side was adjusted to it by numerous malleable-wire ties. A strut, fixing the correct width, was placed near each tie-wire.

Vertical cleavage planes were made in abutments and retaining walls, separating those parts of the wall having different load intensities on the foundation. A cross-partition was built in the form, and the concrete was finished against it. Before building the adjoining part of the wall, the partition was removed, and tar roofing paper was nailed securely to the exposed surface, thus effectually preventing cohesion of the fresh material to the old.

Kaw River Piers.—The three piers of the Kaw River Bridge are of concrete, of the same materials and proportions as used for the pedestals.

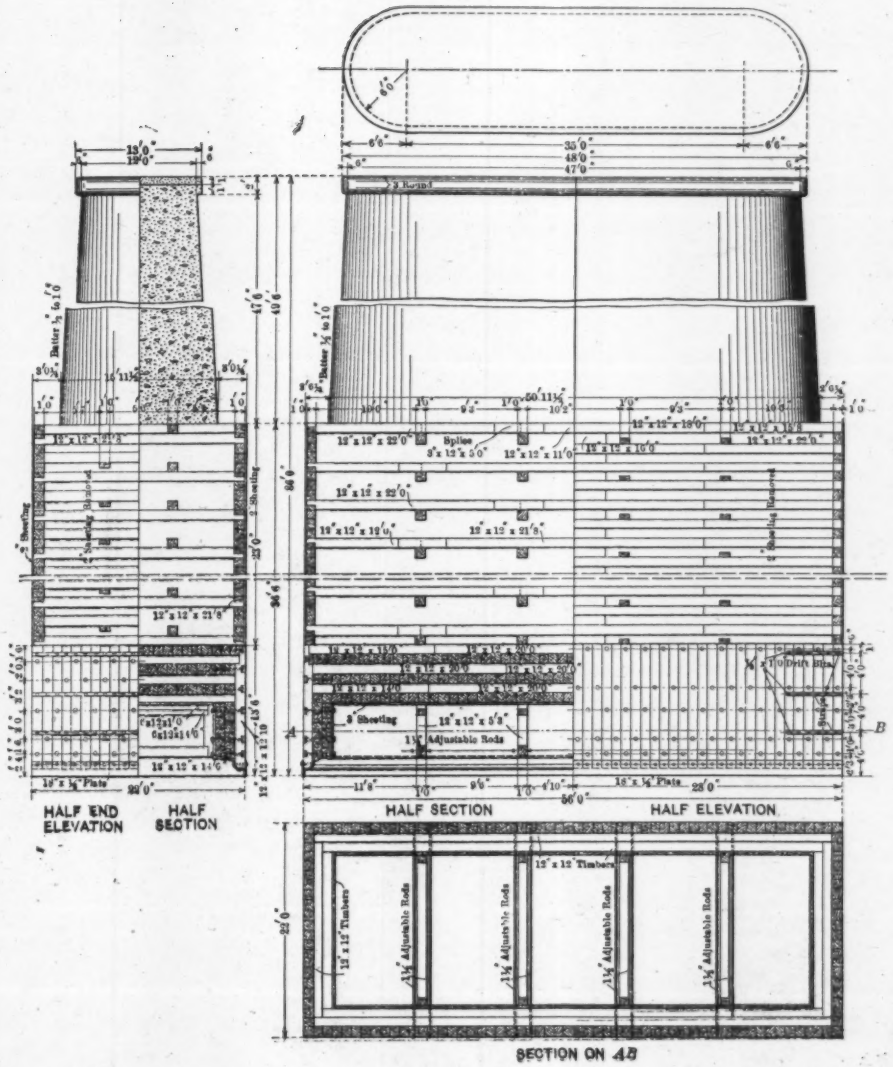
The west shore pier rests on a ledge of rock about at low-water level, so that the construction was very simple, and the work was done in the dry. Light sheet-piling was driven around the pit, which was excavated by hand. Hand-pumps were sufficient to take away the seepage water, and the bed-rock was easily exposed, so that its surface could be cleaned, washed, and prepared for the concrete. The base of the pier was built up nearly to the ground surface.

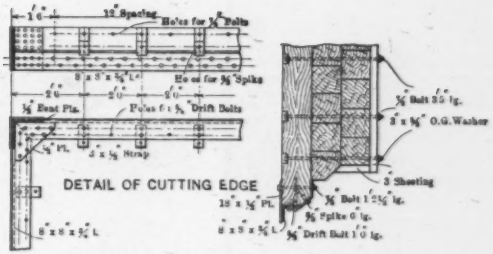
The other two piers also rest on bed-rock, and both were sunk by using pneumatic caissons. The caissons were similar; that of the channel pier was built on shore and floated to place, and that of the east shore pier was built in place.

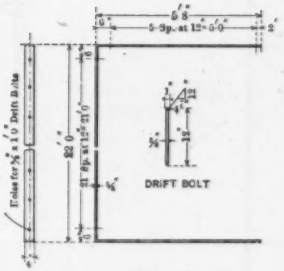
The caissons were of timber and had steel cutting edges. Timbers, 12 in. square, of Oregon fir and Southern pine were used; and were thoroughly bolted and drifted together. The walls of the chamber were 3 ft. thick, and the roof was 6 ft. thick. Every other stick of the upper course of roof timbers was omitted, to give bond for the concrete.

The outer timbers were vertical, and each was fastened by at least two bolts extending into the chamber, besides drifts. Horizontal timbers were lapped at the corners, log-house fashion, thus avoiding extensive framing. The chamber was sheeted inside with 3-in. planks,

PLATE V.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1120.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY.







DETAIL OF 4″X⅜″ STRAP

DETAILS OF PIER 2, KAW RIVER BRIDGE
6TH STREET VIADUCT, KANSAS CITY.

beveled for caulking, and fastened with boat spikes driven through bored holes. Every bolt, and the shank of every spike and nail projecting into the chamber, was carefully wrapped with oakum before being driven entirely home or finally tightened up. On the outside all bolt heads were countersunk, thus leaving a smooth surface. Two cross-bracing trusses, with adjustable tie-rods, stiffened the walls of the chamber.

A false bottom was put in the caisson for the channel pier, in order to float it to position; and, for this pier, a light tramway, on two pile bents supporting a standard-gauge track, was built from shore to pier site for conveying materials. Concrete was mixed on shore and carried out on push-cars in skip boxes. A derrick barge handled the boxes and the timbers for the crib.

The crib, above the caisson proper, was of one thickness of 12-in. timbers sheeted outside with 2-in. plank nailed on vertically.

The usual man and material shafts, with air pipes, blow-out pipes, etc., were installed. The locks used were very simple, being made by merely putting cast-iron diaphragms with drop-doors at each end of a section of ordinary shaft. The one objection to locks of this kind is the necessity of an inside lock tender, and this is not entirely a fault, as men are much less likely to be "plugged" by a lock tender with them in the lock than by a man outside who operates without definite knowledge of conditions inside.

The material excavated was sand, with occasional deposits of silt, and very many boulders, so that most of the material was blown out dry. Numerous relics of the great flood were found in the river bed, indicating deep scour. The metal, notably some street car rails and some parts of steel bridges, was carried down and embedded in the concrete.

Considerable blasting was necessary to prepare the bed-rock for piers. A ditch 3 ft. wide was blasted out around the entire perimeter of the cutting edge, so that the concrete is at no place less than 2.5 ft. below the bed-rock surface. The middle portion of the rock was not removed, but was washed clean, as also was the entire inside of the chamber.

Concrete for filling the chamber was of standard proportions, and was mixed outside and locked into the pier through material shafts 2 ft. in diameter. It was placed in the dry, and carefully tamped to place,

especially against the ceiling, the boulders and pebbles carried down being mixed in. When the remaining space had been reduced to such small dimensions that the men could no longer work, the air was cut off, all doors were allowed to swing open, and into all shafts a thin grout was quickly poured from above until it was some feet above their lower ends. Ordinary concrete was deposited to fill them.

Coffer-dams were not used, but the shafts were built up as soon as the crib was of the estimated required height, and from this point down the sinking was handled very carefully so as to keep the shafts in correct position. For the last 16 ft. of sinking for Caisson No. 2 the shaft moved out of position only ½ in. at each end, and that in opposite directions.

The sinking of Caisson No. 1 was complicated by the presence of a 30-in. high-service water main lying so close to the pier site that the caisson had to be rotated to clear it. Pile bents were driven under the pipe for a support, but these went down, and a sling was put around it and passed up over an **A**-frame to a distant anchorage. All attempts to rotate this caisson to its correct position after it was below the obstruction failed.

These caissons were models in construction; they were well lighted by electricity, and constant and adequate air pressure was maintained. Natural gas was available for fuel.

For Pier 3 the shaft form was of 2 by 6-in. matched dressed flooring, which was placed horizontally on the sides of the pier and fastened to vertical studs, but on the curved ends it was placed vertically and drawn into position by ⅝-in. round iron hoops. These hoops were passed through holes in the vertical studs at tangential points and drawn tight by nuts threaded on their ends. Interior circular templates were placed near the bottom and at the top of each 10-ft. section of the end staves to hold them to position, but were removed as the concrete was built up. Ties of heavy malleable wire were put across between pairs of studs, and twisted tight to bring the sides to proper place. These ties were placed with such frequency as to hamper the placing of concrete, but could not be dispensed with. The building of a 10-ft. section of form and of filling it with concrete alternated, so that the pier was stripped below as it was built up above. Forms of this kind are not considered entirely satisfactory, being easily warped and distorted, and often delaying the concreting for readjustment.

The forms for the other two piers were made of the same planking, in 8-ft. lengths, but this was placed vertically all around the pier. Rigid frames, 4 in. thick, with curved ends of built-up plank nailed and bolted together, were spaced 4 ft. apart, and supported the sheeting. Five ½-in. tie-rods held the sides of the frame together, and cross-bracing at each end and on each side provided against lateral displacement.

The lower edges of the copings were formed to the required radius by sheet-iron corner moulds, and the upper edges by the manipulation of a curved trowel. The tops of copings were brought to a smooth level finish by floating with mortar before the concrete had set.

3.—Field Engineering.

The field engineering included surveys, pier location, inspection, and general supervision of all construction.

The alignment of the viaduct was governed somewhat by property boundary lines. The structure lies in three tangents, aggregating about 6 900 ft., and three curves, each of about 1 900 ft. radius, and totaling 1 500 ft. in length.

Exact surveys and final measurements were made to locate street crossings, railways, buildings, etc., before much work was done on detailed drawings. Base lines, paralleling the chosen center line tangents were carefully marked on the ground, and the angles at their intersections determined. Points on base lines, about 400 ft. apart, were marked by large wooden hubs driven down flush. In the boggy, quaky ground these were 4 by 6 in. and from 3 to 5 ft. long, but in the firmer ground not so large. A piece of sheet copper was nailed on top of each hub, and the exact point was indicated by the intersection of two scribed lines. Each main hub was carefully referenced by marking two intersecting lines. Exact distances from hub to hub were determined either by direct measurement or by triangulation. About one-third of the entire length was triangulated. For the systems of triangles, suitable base lines, usually intersecting with the main line, were marked and measured. Measurements were made as follows: Between two main hubs, 2 by 4-in. marking stakes were solidly driven down, about 98 ft. apart; between these, at about 6-ft. centers, small supporting stakes were placed. All these were sawn off or driven down to conform to the grade from main hub to main hub. When necessary a narrow ditch was dug to contain the stakes, it

being desired to support the tape on them and not on the ground. A 100-ft. steel tape, which had been compared with a standard to determine its constants, was used. The required tension was secured with a spring balance, and the distance was marked by thrusting into a marking stake a knife blade with its sharp edge at the desired graduation. The knife was left there and measured from in placing a second knife ahead, and so on until the second main hub was reached, and there the tape was read to an approximate thousandth. Thus any number of repetitional measurements could be made without the confusion resulting from making each intermediate point with a non-erasable mark. About six measurements were made for each hub distance. The extremes varied from the mean about 1 in 80 000. The base line at the Kaw River was conveniently measured on the deck of an adjacent railway bridge, and the triangulation points there consisted merely of punch marks on the end floor-beams at fixed shoes.

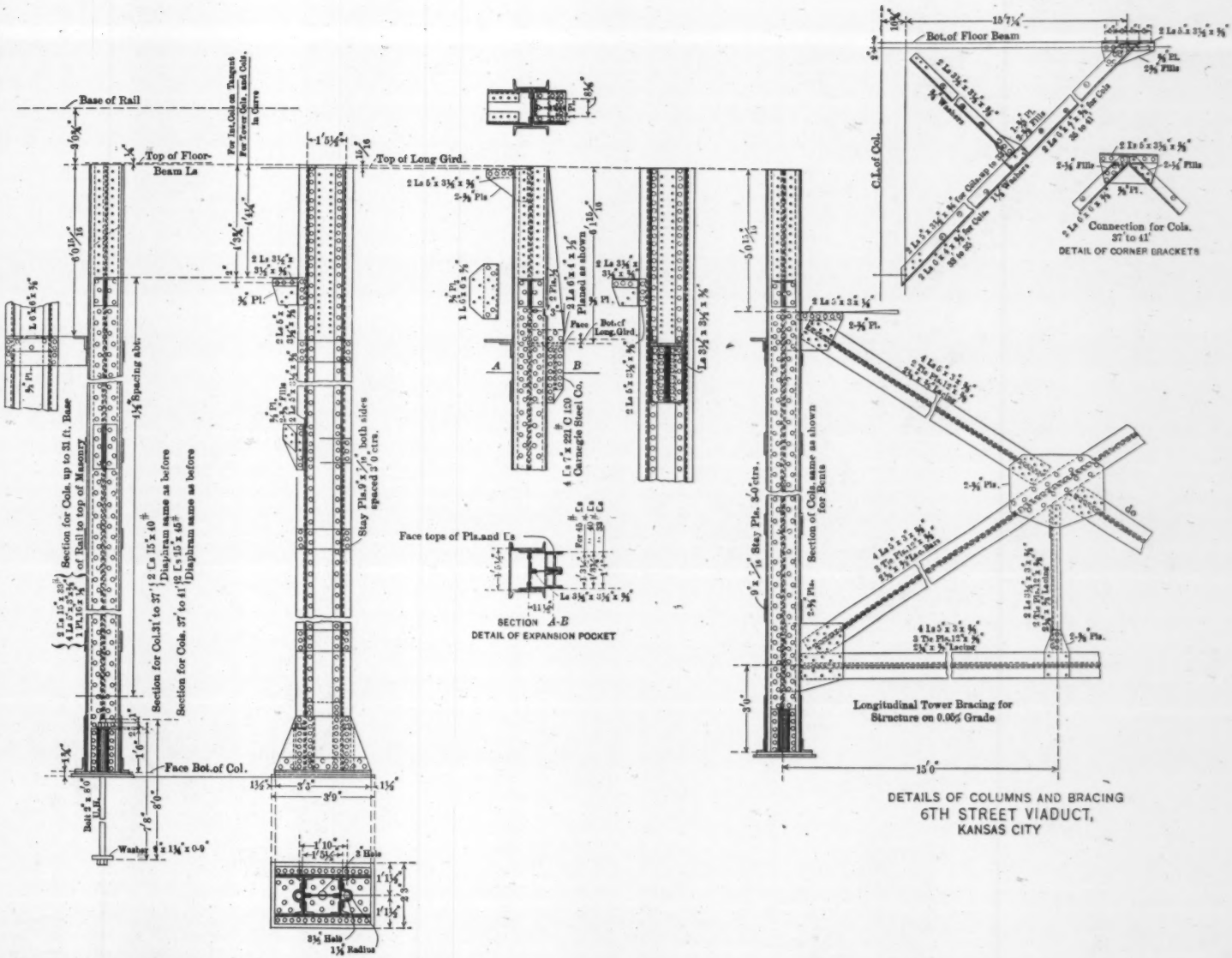
The measurement of angles was done by a method of repetitions and reverses which eliminated instrumental errors in the mean of each series of readings. Enough readings were taken at each point to make the triangles close within 1.5″. The largest triangulated distance was 1 131 ft., and was determined by two triangle systems, so that the independent calculated lengths checked within $\frac{3}{32}$ in. The difference in the elevation of the two end points was 65 ft.

After all distances had been determined, the station and plus of each hub was calculated, and locations for the pedestals were measured readily from the nearest hub.

For the location of river piers, the true angular distance from each base line was calculated, and a temporary fore-sight was established by a single carefully-turned angle which was then measured exactly. The distance of the temporary fore-sight from the true line was calculated, and the point in the hub was then moved the necessary distance ($\frac{3}{4}$ to $1\frac{1}{2}$ in.). When the two reference lines and the bridge tangent were projected to the piers they intersected within a $\frac{3}{16}$-in. circle.

For pedestals, and for piers where possible, reference lines at right angles to the tangent were marked by several hubs well driven down. The longitudinal center lines of each row of pedestals were also suitably marked at intervals. Elevations along the entire line were determined, and numerous carefully checked bench-marks established.

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS
VOL. LXV, No. 1120.
HOWARD ON
THE SIXTH STREET VIADUCT, KANSAS CITY.



DETAILS OF COLUMNS AND BRACING
6TH STREET VIADUCT,
KANSAS CITY

The inspection of construction embraced every detail, and daily records of the progress of each kind of work were prepared. For instance, not only did an inspector watch the placing of concrete, but a second man watched the entire process of mixing. There was an ample number of men to make inspection of stone, timber, cement, sand, pile-driving, forms, concreting, erection, riveting, painting, paving, decking, etc., as has been noted previously.

4.—Erection of Superstructure Metal.

About 88% of the total length of the viaduct is of practically duplicate panels, and, in all details of construction, particularly in the erection, this characteristic feature permitted economic handling of materials and labor.

Trestle Erection.—The traveler used for the ordinary structure consisted of a pair of derricks mounted on a frame arranged to run on track rails laid on the longitudinal girders, the masts thus being 33 ft. from center to center. The details of this traveler are clearly seen on Fig. 2, Plate IV. The masts were double, one stiff and one seated to turn. Each was a 14-in. square timber 40 ft. high, and the booms were built up of spliced timbers to 14 by 26 in., and 60 ft. long. The sills and cross-sills were 8 by 16 in. All connections were made by steel plates, and steel derrick fittings were used throughout. The equipment consisted of two 2-drum, 4-spool, hoisting engines of 20 h.p., each handling one boom with its lines and runners. In the ordinary structure the members varied in weight up to 10 tons, which one boom could easily handle.

A service track was laid between the pedestals nearly the entire length of the viaduct, and the steel was unloaded alongside. The intention was to distribute immediately every piece to its proper location, but, owing to the operations of the substructure contractor, this could not be done, and it was necessary to store and redistribute them. The lighter metal was unloaded by a locomotive crane, the larger girders by yard derricks, or by hand upon cribs.

It thus followed that a considerable part was erected from metal already distributed, but also a large part directly from the cars as they were run down from the yard. There was little difference in the rate of erection in the two cases, the former being retarded by the necessity of removing superposed pieces and hitching to material some-

what indiscriminately piled, and the latter by reason of the turbulent operation of service cars over the temporary tracks. Fig. 2, Plate IV.

The pedestals were finished ½ to ¾ in. low, and four 2 by 4-in. oak shims of suitable thickness were used to bring each column to the desired elevation. The anchor-bolts had been located so accurately that they sufficed to line up the columns, additional marks being unnecessary.

The usual order of erection per panel was as follows: Two columns were set up at once, one by each boom, and braced up by the anchor-bolts; the connecting floor-beam was placed on its shelf-angles, and bolts were put in one-third of the open holes. The cantilevers were then put up and bolted, after which the main girders were raised singly, both booms being used for the heavier girders. The stringers were then laid on top, to be distributed later, and the traveler was immediately moved forward over the new panel, ready to set the next columns. The maximum day's run was six panels, about 275 tons.

The light material in the deck was put in place in conjunction with the riveting, the stringers by using timber dollies, the bracing, etc., on a push-car running on the flanges of two adjacent stringers. However, as the traveler proceeded, it placed all sway bracing, tower bracing, and laterals.

Certain long and heavy girders required some variation from the usual order of handling. Where the columns ahead were beyond reach of the booms they were raised by a gin-pole or a gallows-frame, which then served to lift one end of the girder, the other end being lifted by the traveler. The longest girder in the structure (107 ft.), weighing 62 tons, was erected in this manner, the gallows-frame consisting of a pair of 18-in. square timbers, 60 ft. high, connected with bracing and caps, equipped with 4-sheave steel blocks reeved with steel cable. Fig. 1, Plate VII.

Because of buildings and railway tracks under the eastern seven panels, the girders could not be handled from below, but it was necessary to run the traveler back, free of obstructions, pick up a girder, set it transversely on the traveler track, move it forward by picking up, booming out, and shifting the traveler, and finally to swing it around longitudinally to its final position. The columns in the buildings were dropped down through holes cut in the roof, with small trouble to the occupants, who, however, did not object to stepping

outside for a few minutes while each girder was being manipulated above their office rooms. The girders immediately adjacent to Bluff Street extend through slots or pockets cut in the retaining wall which supports the street, and rest on pedestals 15 ft. back of, and thus entirely independent of, the wall. The tracks below these girders carry all the passenger traffic into the Union Depot, therefore unusual caution was necessary in erection to guard against possible interruption of train service.

Erection of Truss Bridges.—The falsework consisted of 7-pile bents every 30 ft. under the panel points, and 5-pile bents midway between, the piles being well driven into the river bed. Four 8 by 16-in. stringers were placed under each truss, and a traveler track was supported on two runs of 12 by 12-in. timbers. This traveler was composed of two derricks rigged together, and running on a track between the trusses in the plane of the bottom chords. Beside the traveler, also between the trusses, a standard-gauge track was laid for moving out the metal.

After fastening down the fixed shoes, the bottom chords were laid on the camber blocking for the full length of the span. Beginning at one end, the web members, chords, floor-beams, and laterals, were erected in regular order.

The parts of these trusses are heavy, and the connection plates large and stiff, so that fitting together the various members was tedious work. A clearance of $\frac{1}{16}$ in. was provided, but, in a member 22 in. wide, made of four $\frac{3}{4}$-in. angles and a $\frac{3}{4}$-in. plate, which had to be entered from 5 to 6 ft. into a second member equally stiff, slight irregularities would cause binding. Practically every diagonal and chord section had to be pulled to place with a runner.

It was apparently impossible to do all the field riveting before the time of the expected washout of the falsework, and, in order to support the spans in that contingency, drift-pins were tightly driven into one-third of the open holes, and fitting-up bolts into one-third of the open holes of all main connections. Fortunately, it was not necessary to trust to this expedient, as the expected high water did not come. It is evident the spans would not have fallen, but experience showed that there would have been a slight movement at each point, which, in the aggregate, would have caused trouble.

The trusses were set carefully to correct the camber which was

maintained until the erection was complete. With the passage of the time required for riveting, the falsework settled at points, permitting the entire truss to settle. Attempts were made to jack up the trusses, but the only result was a pushing down of the piles under the wedges. A regular camber of about one-half as much as estimated was finally secured. With the complete dead load and such live load as has been on the spans, the bottom chords are just about straight. The serious consequence of this settlement, however, was in the tightening of the drift-pins. These were not taken out until the rivets had been driven in all the adjacent holes, and then it was found there had been enough movement to pinch them tightly in place, requiring much energy and effort to remove them. Most of these pins had been driven toward the inside, and the cramped quarters prevented efficient striking to back them out. Many had to be drilled out, and, as they were of very hard steel, it was difficult to keep the drill point on the pins and prevent it from running off into the softer metal.

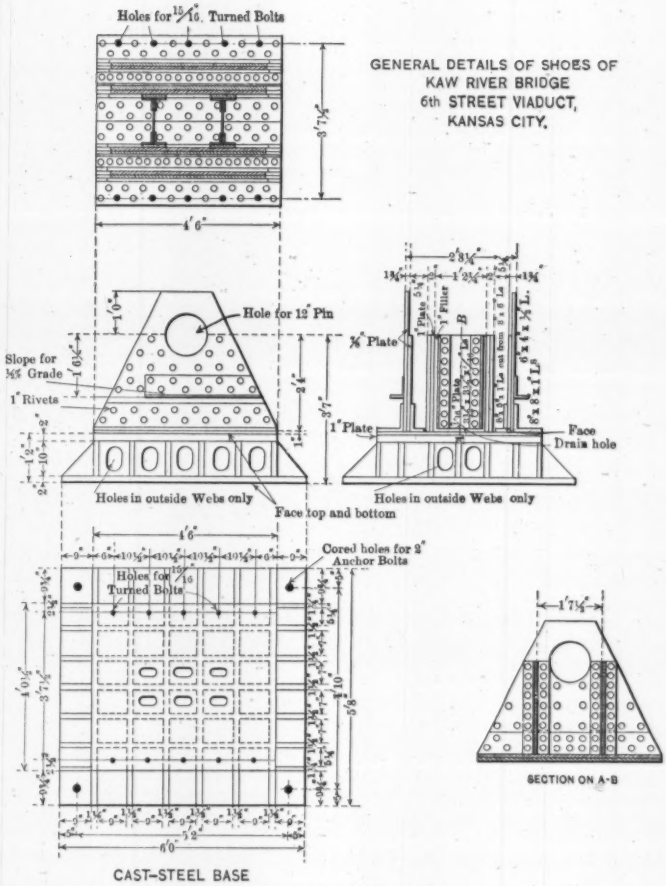
The lower chord splices of these spans were riveted first, and then the ends of the diagonals; but the upper chord splices and the intersection points of the diagonals were not reamed or riveted until the span had been swung.

The similar trusses for the 147-ft. span were erected on a falsework of framed bents set on the ground. There was no trouble in maintaining the full camber in this span; and, as the members are much smaller than for the Kaw spans, their erection was quite simple.

Riveting.—The contractor sought and received permission to ream all field holes in the field, thus making it unnecessary to assemble in the shops; and the metal as shipped out was merely sub-punched. Because of this, and since, for the unrestricted movement of various gangs, it was desirable to keep them separated, a very large amount of scaffolding was placed and retained for some time, but it was of simple character.

The joints were first thoroughly fitted up, and bolts were put in half of the holes; the other holes were then reamed, the bolts were shifted to the reamed holes, and the reaming was completed.

Each joint was thus fitted twice before the rivets were driven, but trials of reaming and then riveting half a joint, and then reaming and riveting the remaining half resulted in the interference of the various gangs, and better progress was made by the procedure given.



FIXED SHOE FOR WEST SPAN

FIG. 5.

Neglect in fitting up caused the majority of cut-out rivets, for if the plates were not drawn tightly together small particles of reamer drillings would work in between them and cause successive rivets to loosen those previously driven. Natural conditions were seized upon to assist this work; for instance, the expansion joints were regularly rammed full of steel wedges, thus gaining the thrust of expansion to force together columns and girders.

The reaming was done with Little Giant, No. 0, pneumatic drills, equipped with 4-flute, tapered reamers having barrels at least as long as the length of the hole. Water, with a little soap, was used as a lubricant, and was kept constantly flowing on the bits. A knuckle-joint was tried for points where the reamer could not be held normal to the metal, but gave indifferent results; so, when unavoidable, holes were reamed on a slight bevel, but they were required to be perpendicular to the direction of stress. There was some trouble in making all holes of the same diameter, for the reamers had to be held very steady to accomplish this. The ordinary run for two men was from 200 to 250 holes, $\frac{7}{8}$ in. in diameter, with a 3-in. grip, or equivalent, in 8 hours. The smooth, clean holes permitted the rapid driving of rivets without further drifting or fitting, and without the delay incident to entering hot rivets into imperfect holes. This was highly essential, especially in such cases as the lower chord splices of the Kaw River spans, where the rivets are 1 in. in diameter and have a 6-in. grip.

Cleveland air hammers, with a 9-in. stroke, operating under a pressure of 110 lb. were used throughout. Grip dollies were required wherever they could be utilized, and hand-forges were used almost exclusively. With one heater and two riveters, 550 rivets, of $\frac{7}{8}$-in. diameter and 3-in. grip, were driven in 8 hours.

The greater part of the riveting was open, easy work, although there were marked exceptions. The rivets in expansion pockets required special, crooked-jamb, joint dollies, in order to pass around the end stiffeners and rest on the next adjacent stiffener.

The bracing of the motorway stringers was connected by $\frac{3}{4}$-in. rivets driven by hand without reaming—in fact, it was found that the lug-angles on the beams were set so high that the rivets were under the flanges, thus precluding the use of an air hammer. Very fair results were achieved, but it would have been much better to have had wider lug-angles.

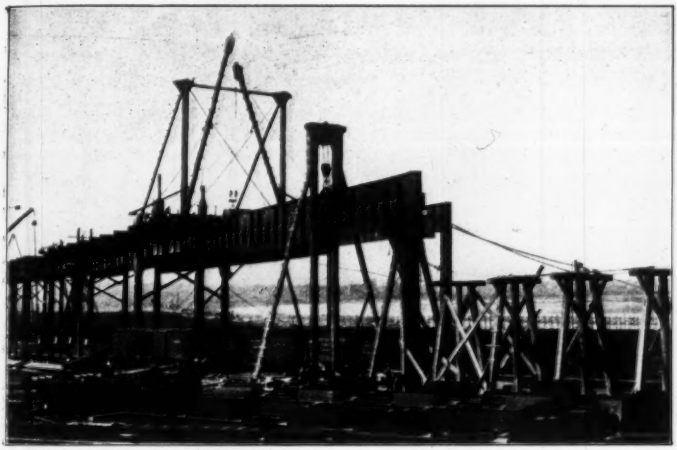


FIG. 1.—GALLOWS FRAME USED IN THE ERECTION OF LONG GIRDERS.

FIG. 2.—CONCRETE FLOOR-TROUGH AND COVER STYLE OF FORM.

In the main structure there are about 300 000 field rivets. While accurate costs are not available, the following figures are a close approximation: Reaming cost 7 cents per hole, of which 5 cents was for labor and 2 cents for plant and tools; driving cost 8½ cents per rivet, 8 cents for labor, and ½ cent for plant and tools. It is possible that too much of the plant cost is charged to reaming and too little to riveting. There were reported cut-outs of 3⅓%, but apparently all the rivets were not accounted for, and 5% would be nearer the true figure.

Grouting Columns.—The space below each column base was filled with grout. A cement mortar of ordinary consistency was first prepared and a little daubed under the edges of the base plate around its entire perimeter and finished off neatly. After a day's time there would remain a tight dam all around, pierced, however, by a small hole on each side. The nuts and washers on the anchor-bolts were temporarily removed. Grout, diluted to a pouring thickness, was made in the proportion of about 1 of cement to 3 of fine sand containing a little clay. Several bucketfuls of the mixture were poured, as quickly as possible, into the anchor-bolt holes and the central hole until it ran out of all the "air holes," and then these were promptly daubed up. Mortar was added to fill entirely the side pockets; the foot of each column was filled with concrete finished off with mortar.

This method was highly satisfactory, as it gave a full and even bearing under the entire surface of the base plate, as was proven by lifting up columns which had been grouted.

For large solid castings, as under the Kaw spans, grout could not be used. The bases were set on steel shims about ¾ in. above the concrete. A stop was put at one side, and small quantities of almost dry 1:1 mortar of cement and sand were put under the opposite side, pushed clear across, and rammed with a piece of ½-in. plate. The entire space was filled from one side, thus insuring the absence of voids.

Had holes through the casting been provided, the former method could have been used with superior dispatch and ease.

Painting.—The steel received one coat of paint in the shops, and presented a good even coating when unloaded at the site. As it was erected, all abutting parts and surfaces inaccessible after erection were coated with a heavy paint cement daubed on in generous measure.

Upon the completion of the riveting of a section of the viaduct, the entire surface of the metal was examined, carefully cleaned with wire brush and scraper where necessary, and the heads of field rivets and all areas where the shop coat was scraped off or unduly marred were touched up with paint. Reaming borings added considerably to the ordinary work of cleaning. When these spots had dried, the first field coat (of green paint) was applied. The paint was delivered ready mixed, and was of rather stiff consistency, requiring, for proper application, small amounts thoroughly rubbed on by the vigorous use of a stiff-bristle brush held almost normal to the surface.

When this coat had dried, requiring a varying time, but determined when it became impossible to "rub off" the dried surface skin and expose wet paint below, the final coat was put on. The same care was used in application, and the entire structure was finished up progressively.

The inspectors then examined the work carefully, and remained with the finishers until the painting was acceptable. Small hand-mirrors were used to reflect the sunlight and illuminate dark corners and shadowed areas, thus exposing neglected patches almost impossible to locate by any other means. In many places, too, as, for instance, inside a column, it is much more convenient to observe the reflection in a mirror than to attempt to view the parts directly.

No painting was done in wet, or wet and freezing, weather, but the work was never stopped because of low temperature, provided the surface to be coated was dry and free from sleet or ice.

5.—Viaduct Floors.

The general details of the reinforced concrete floor are shown on Fig. 3. The average slab thickness is 6 in. above the stringers, the stringers being entirely encased, both to protect them from the smoke of locomotives below and to secure the added strength of the filled haunches. The amount of reinforcement is considerably in excess of actual load requirements, in order to provide for the temperature stresses of the concrete. It consists of transverse and longitudinal bars, near both the upper and lower surfaces of the slab, of extra bars over the cross-beams, of numerous vertical bars in the curbs, and of wire netting around each I-beam stringer. To support the hand-rail

posts, bolts project through the sides, the lower bolts passing through the I-beams.

In order that the usual paving could be continued without interruption over the points of expansion, a flexible connection was devised consisting of U-shaped plates bolted side by side. Thus a sort of bellows trough is formed, about 6 in. high and 2 ft. wide. One such bellows extends across the roadway at each expansion joint of the stringers, and is riveted to each stringer on both sides of the opening. The plates are thin enough to spring easily under the movement of the steel. The concrete is finished up to each side of the bellows, and fastened to it by bolts. These troughs were to be filled with a rather soft asphaltic material, the flow of which would accommodate it to the varying position of the containing plates.

The specifications provide for concrete made of cement to be supplied by the company; of clean, hard, broken stone "free from all dust, dirt, shale, rotten stone, or other objectionable material", to pass a 1-in. ring; and of Kaw River sand in the proportions 1:3:5 by volume, loose. To provide a bond for the contemplated wearing surface of asphaltum (which was to be laid without the usual binder course) it was required that clean stones be sprinkled on the surface of the wet concrete, and be tamped into it enough to embed them partially, yet permit them to project a little. Other principal specifications were: That the entire thickness of the floor was to be deposited in one continuous operation so as to avoid laminations or dry horizontal planes between consecutive layers; that all surfaces were to be finished neatly to the exact required dimensions; that all bolts and embedded castings were to be set to exact position; that reinforcing bars were to be spaced accurately; and, finally, that the contractor was to clean all stains and concrete droppings from the metal work.

The motorway stringers at the side of the roadway were used as a runway for the concrete mixing equipment. A standard-gauge track was laid on these stringers for some distance, to accommodate the contractor's plant, which he placed on several railway flat cars. This plant consisted of two large elevated hoppers, connected by a cable carrying a conveyor bucket, and a belt conveyor for handling finished material, and similar appliances. Much time was consumed in the arrangement of this equipment which was soon found to be unsuitable. The company's engineers then designed a new and simple plant, the

old elaborate one was dismantled, and five of the smaller plants were quickly built and put into successful operation.

The plant is shown in Fig. 1, Plate VIII; it consisted merely of a platform of timbers, mounted on wheels to run on the motorway stringers, supporting a mixer, a derrick, and an engine. The derrick raised the aggregates from below with a skip box, and a metal-lined chute extending over the roadway received the concrete from the mixer. The total cost of one plant, including all fittings except the mixer and the engine, was $700.

Forms.—Several arrangements of forms were tried, and, as these made up a considerable part of the floor concrete cost, they merit special attention.

The contractor first furnished all-metal forms, made of corrugated sheets pressed to shape and covered with light sheet iron. They were made in sections 5 ft. wide and about 5 ft. long, the longitudinal junction of various sections being immediately under the stringers. Along each of these lower edges there was fastened a small angle having holes in its vertical leg. Keys were provided to pass through these slots to fasten adjoining forms together laterally. Provision was made for the forms to lap about 1 in. longitudinally, but there was no way of fastening them together. To support these forms, three joists were placed from cross-beam to cross-beam, and three sections of form were laid thereon. Numerous imperfections were soon evident. The form had almost no lateral stiffness, each unit having an individual shape. At best, the surface presented was not true; the correct distance was not left on each side of and below the I-beams; there were openings and cracks at the transverse joints, as these could not be made to lap as intended; many pieces had to be held during adjustment; and, also, the forms were easily displaced by the ordinary walking about and working on them.

Owing to the non-delivery of the metal forms, the contractor had already begun to build wooden forms of much the same type, that is, of sections to meet under the stringers and of such length that three were put between adjacent cross-girders. They were built of 1 by 4-in. frames spaced 18 in. from center to center, sheeted with ½-in. boards, and covered with No. 14 sheet iron. These forms were hung on the stringers with two ¾-in. round, soft iron rods on each side. After adjusting the form to position, the hangers were bent around the upper

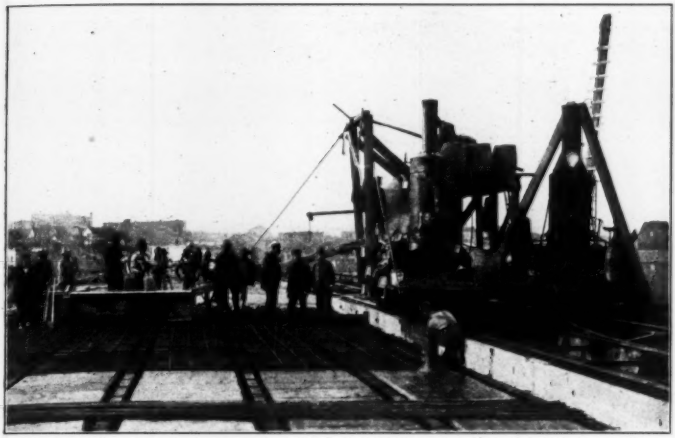


FIG. 1.—ARRANGEMENT OF CONCRETE PLANT FOR LAYING FLOOR.

FIG. 2.—FINISHED FLOOR OF MULBERRY AND HICKORY STREET APPROACHES.

flanges of the I-beams. To remove the forms, the hanger rods were cut with heavy clippers close to the sheeting. These wooden forms were light and stiff, but they, too, showed objectionable defects. There were no means of really fastening the various units together, and there were presented, not only the transverse cracks of the metal forms but also a longitudinal crack under each stringer, and these latter were almost inaccessible for inspection or adjustment. Nothing was found to prevent water and cement from leaking out.

The company's engineers then designed a form, and the use of the others was discontinued as soon as they could be replaced. The finally acceptable form consisted of two parts: U-shaped troughs which were placed under the stringers; and covers which extended from trough to trough between stringers. They were of wood, 1 by 4-in. frames, with ½-in. or ⅝-in. sheeting, and covered with No. 16 or No. 18 sheet iron. The troughs were 13 ft. 10 in. long, or just enough to go easily between the cross-beams; the covers were of various convenient lengths, from 4 to 6 ft.; and the outside curb faces were 18 ft. long. To support the troughs, ¾-in. round iron hangers were used, a loop-shaped piece being put around the 2 by 4-in. sill on the bottom, and each leg being bent over the upper flange of the I-beam. Four or five ties were used for each section. To hold this trough in correct position and prevent rotation, briquette spacers were used. These were made to fit the lower curve of the form, and had a shoulder on each side at the top to fit the lower flange of the I-beam.

A nest of wooden moulds was provided, and briquettes of sand and cement were made in advance of the need, so as to be thoroughly hardened. One briquette was placed at each end of each trough (thus closing it against leakage) and one under each hanger. When the hangers had been drawn reasonably tight and bent over, the trough was so rigid that a man could walk along one edge.

The cover forms, it was found, could be handled with a lap at one end, and, when the troughs were in position, could be rapidly laid, producing a continuous unbroken surface. The small remaining vertical hole (8 by 12 in.) at each floor-beam was covered with a piece of No. 8 sheet iron. The forms for the outside faces of the curbs were made of ⅞-in. plank. These were stood on the already erected curb trough, adjusted to position, and nailed lightly. A horizontal hanger held them laterally, and they could be made truly vertical.

These forms proved to be highly satisfactory, presenting smooth, rigid, correctly-shaped surfaces, with very few chances of leakage. They were easily altered to meet conditions, and stood rough treatment well. Their erection and removal was simple, any part being handled by two men without extra scaffolding. Fig. 2, Plate VII.

The briquette detail was one of the most advantageous features, and was developed by a prior use of wooden blocks, which gave trouble by breaking out the concrete as they were removed with the form. The cement briquettes remained tightly in place, and panel after panel of floor was built without a place to touch up below the beams.

The cost of building 600 lin. ft. of these forms for the 30-ft. roadway was:

Lumber	$1 160
Sheet iron	650
Nails and sundries	165
Labor	1 300
Contractor's profit	325
	$3 600

The first cost was $6 per lin. ft., or $0.20 per sq. ft. between curbs. Forms used five times cost 4 cents per sq. ft., or $1.30 per cu. yd. Many of these forms were used oftener than five times. The fact that they were built in great haste doubtless increased their cost somewhat.

Making and Placing Concrete.—Owing to difficulty in securing a sufficient quantity of 1-in. stone from local crushers, the use of flint screenings from zinc mines was permitted. Although this material is clean, hard, and sharp, and is somewhat graded in size, needing but little sand, there are objections to it. A few moments after it had been deposited quite wet, the water appeared to drain away, leaving the upper stones clean and dry, without any mortar coating whatsoever, and the entire mass appeared to be inert and "dead". This appearance was evident even while the concrete was still green enough to be dug through with a stick. After several days the main body became hard and compact, but on top there was perhaps ¼ in. of loose stones. Various mixtures were tried, to see if the concrete could be made to remain "quaky" for a time, but it could not be done. Yet experiments on the

tensile strength gave creditable results, and investigations by cutting through slabs thoroughly set showed the concrete to be very hard. About half the floor concrete was made of this flint, and the remainder of crushed limestone. The latter, of course, could be flushed with mortar, and gave normal results.

All concrete materials were delivered on the ground under the viaduct. Skip boxes were filled with a batch of correctly proportioned aggregates and raised to the mixer above. The concrete was dumped into a trough and slid down to be distributed over the floor. To expedite the filling of the small space below the I-beams, cement mortar was used to a height of about 2 in. above the lower flanges.

The reinforcing bars were spaced and held in position as follows: Round ¾-in. iron rods were laid longitudinally on the forms, about 5 ft. apart, and on these the lower transverse bars were placed. The lower longitudinal bars were then laid in position and fastened at numerous intersections with wire ties. When some 3 in. of concrete had been spread, a rope was fastened to the ends of the ¾-in. rods and a strong pull slipped them out longitudinally from the finished work. A very minute inspection of the whole floor failed to show a single instance of a lower bar exposed.

Combined supports for the upper bars and guides for finishing the upper surface consisted of 2-in. joists having small angles fastened on each side and a series of hooks in the lower edge. They were placed longitudinally with the structure and 10 ft. apart, and were held on small blocks resting on the stringers. The hooks supported the transverse upper bars in correct position. A gauge, sliding along the angles, gave the desired surface shape to the concrete.

The concrete was first tamped with a large flat tool and, after hardening somewhat, was retamped with a corrugated tool. The latter was made of wood with parallel cleats, ¾ in. wide, 1½ in. deep, spaced 2 in. apart. The cleats were sharpened slightly. With proper manipulation this produced light corrugations, in varying directions, of $\frac{3}{16}$ to ¼ in. variation, thus giving a satisfactory bond for the asphaltum. Scattering loose stone and tamping it in was tried, but the former roughening was superior. Neither method gave entire satisfaction where flint was used.

The plants were moved forward as each section of about 15 ft. of roadway was completed. On an average, the five plants placed from

300 to 350 lin. ft. per day; the best day's run for one gang was 98 lin. ft., or 90 cu. yd.

The floor was finished level to the edges, and the curbs were built later, in the usual way. Bars projected upward into this curb every 12 in. and bound it to the body of the work.

With all types of forms, and to a pronounced degree with the first designs, there was considerable leakage from the wet concrete. This dripping water carried such a quantity of cement as to leave a whitish deposit on the metal where it drained, and in certain cases spattered down on the ground under the structure. Probably the actual loss of cement was small, but although it was easy to know how much cement went into each yard, or each day's run, no scheme was discovered to ascertain how much actually remained in the concrete. This cement was not cleaned from the metal promptly, and the cleansing later proved troublesome. The plan was to wash the metal with a strong stream of water immediately after the concrete was placed and until it ceased to drip. Adequate pumps and lines were not installed to do this, and finally much cleaning had to be done with wire brushes and scrapers, necessitating some repainting.

The entire concrete floor was built in 67 working days, during which there was some freezing weather. The contract provided for heating any or all concrete materials, but none was heated other than some sand and this merely to thaw frozen lumps. However, wagon sheets were spread over the green concrete each night, and, when hard weather was expected, these were covered with stable sweepings. No bad effects were found on the floor proper, but some portions of curbing were frozen and had to be torn out and replaced. Some of the concrete was laid when the temperature was 31°, and the following days were about the same, yet in 10 days' time asphalt rollers were run over the work.

The 30-ft. roadway contains 24.6 cu. ft. per lin. ft. and the 38-ft. roadway 33 cu. ft. per lin. ft., or an average for both of 0.84 cu. ft. per sq. ft. of area between curbs. The average amount of reinforcing is 2.67 lb. per sq. ft.

The main viaduct contains about 8 000 cu. yd. of concrete. The contract cost, exclusive of cement and bars, was $6.085 per cu. yd. The cost of completed floor, not including paving, was about 36½ cents per sq. ft.

Paving.—The paving is of Bermudez asphalt. The following results are from average analyses:

Specific gravity	1.098	
Insolubles	2.0%	
Soluble in 88° naphtha	63.0	
Soluble in 62° naphtha	85.0	
Penetration at 77° Fahr.	38°	(Dow.)
Fixed carbon on ignition	14.5%	
Ductility	20.	(C. M.)

This hard asphalt was softened by the addition of asphaltic petroleum residuum which tested as follows:

Specific gravity, Baumé	11.25
Boils	83° cent.
Flashes	173.5° cent.

From 18 to 20 lb. of this oil were mixed with 100 lb. of asphalt. The hardness of the resulting mixture at 77° Fahr. varied from 67 to 78° (Dow).

It is rather difficult to secure just the sand desired for asphalt in Kansas City, as the Missouri River sand is somewhat water-worn and the finer Kaw sands contain some loam. Pulverized limestone and cement were added to the sand to give the required 200-mesh material.

The following mix was taken as a criterion, although the specifications gave somewhat more latitude:

Bitumen	9.5% to 11.5%
Sand passing 200	6 % " 16 %
" " 100	8 % " 16 %
" " 80	10 % " 16 %
" " 50	14 % " 34 %
" " 40	9 % " 16 %
" " 30	6 % " 10 %
" " 20	3 % " 7 %
" " 10	2 % " 4 %

The paving material was given close inspection, both at the plant and on the work. Samples, taken daily, were analyzed and sifted, and daily directions were given to the mill. On the viaduct, samples

were taken from 1 in every 5 to 15 loads, depending on the rapidity of delivery, etc.

The rolling was done carefully with 6 and 8-ton rollers. Where necessary, the surface was ironed, with especial attention to securing adequate drainage. The entire pavement was placed in 20 days.

The iron trough expansion joints were filled with the ordinary mixture. At some of these the wearing surface cracked during cold weather, and in the later work thin separators were inserted to make these openings straight across the roadway. These joints are acting as expected, and are considered very satisfactory.

Asphalt was not laid on the new embankments, but a Macadam pavement on a very heavy base was substituted and finished with oil.

The contract cost for the asphaltum was $1.05 per sq. yd., and for the Macadam, $1 per sq. yd. Thus the concrete floor with pavement cost about 48 cents per sq. ft.

Street-Car Deck.—The ties of the street-car deck are of creosoted, short-leaf, Southern pine, 6 by 8 in., laid flat, and spaced 13 in. from center to center. Every fourth tie extends across both tracks, thus serving to brace the whole deck together and support a possible runway. Guard timbers are dapped 1½ in. on the ties and are set 11 in. beyond the gauge. On curves, the ties are on edge, the variation in the depth of the daps giving the outer-rail elevation.

All the timber was surfaced on four sides, and finished to full size. The general creosoting specifications were the same as already given for piles, from 10 to 13 lb. of pure dead oil of coal-tar being used per cubic foot of timber. The dapping, boring, and cutting were done before creosoting. The deck timber cost delivered $45 per 1 000 ft., B. M.

A small derrick car, equipped with a gasoline hoist running on the stringers, raised the material to the decks from its distributed position below. The ties were set to approximate place and the guard rails laid. Wooden rams were used to drive the guard rails down on the ties, to shift the ties to fit the guard-rail daps, and to settle them on the stringers. Then the hook-bolts and guard-rail bolts were inserted, and the deck was tightened to place with socket wrenches on auger handles.

The rails are of 70-lb., Am. Soc. C. E. section, 30 and 33 ft. long, laid with broken joints, fastened to every tie with two ordinary 5½-in. spikes. Six-hole angle-bars were used.

The tracks are bonded by two copper bonds at each joint, and set up with a screw compressor. For each bond, two $\frac{7}{8}$-in. holes were drilled through the rail web. Some of these holes were drilled with hand-ratchets, but as the rails were very hard and the drilling was slow, a machine drill was substituted. This consisted of a $1\frac{1}{2}$-h.p. gasoline engine, set on a push-car, operating a flexible shaft, which was attached to an automatic-feed drill working in a yoke that fitted over the rail. This machine cost only $100, in addition to the engine, and gave creditable results. It would drill the four holes at a joint and move forward in 12 min. A very weak solution of sal soda was used for lubrication.

The cost of laying deck timber was about $5 per 1 000 ft., B. M.; for laying rails about 4 cents per lin. ft. of single track; the work of bonding cost about 50 cents per joint.

The usual overhead trolley is used. The poles are of iron, from $6\frac{1}{2}$ to 5 in. in diameter and 20 ft. high above the ties. They are placed between the two tracks, about 90 ft. apart, and support cross-brackets for the wires.

Hand-Rail.—The hand-rail was designed for strength, simplicity, and architectural harmony with the structure. It consists of square, cast-iron posts, 9 ft. apart, supporting horizontal rails of ordinary pipe, the diameter of the upper one being 3 in. and the other four $2\frac{1}{2}$ in. Each post is fastened by two bolts embedded in the concrete floor, the upper of which has two nuts, in order to provide for the adjustment of the post. The pipe rails pass through holes in the posts, fitting rather loosely, and are joined by standard unions and made continuous. At each post, and for each pipe, two small steel wedges are driven into seats, gripping and pinching the pipe and holding it tightly in place.

Thus the hand-rail could not be erected until the floor was finished, and then the posts and rails had to be put up simultaneously. A small two-wheeled truck was used to set up the posts. The forward end of this truck had a cantilevered fork on which a post was supported as it was drawn on to the projecting bolts. The railing pipes were inserted consecutively and jointed together as the successive posts were erected.

When several hundred feet of railing was in place the posts were adjusted to points set by a transit, thus bringing the upper rail to a true line. The cost of the hand-rail is included in the general schedule price for superstructure metal erected.

Lighting System.—The annoyances of a trip through the bottoms are aggravated by darkness, and an adequate lighting system for the accommodation of night travel was considered highly desirable for the viaduct. Natural gas is available, and was considered, as also were other plans for lighting, but an incandescent electric system was selected. The results are exceedingly satisfactory; the illumination is uniform and not glaring, yet so bright that a newspaper can be read anywhere on the roadway.

Every fourth hand-rail post is extended above those intermediate, and supports a 16-c.p. Tantalum lamp, set in a vertical, moulded, mica socket, surrounded by a glass globe 10 in. in diameter. There is thus a light every 36 ft. along each hand-rail. The lights are uniformly 8 ft. 6 in. above the pavement, and are staggered on the two sides, making one lamp for each 18 lin. ft. of roadway.

The wiring is of the three-wire, 250 to 500-volt system, with two lamps in series, connected in three circuits so that any third of the lamps may be used without changing the factors of symmetry and distribution. Certain main posts have lamps wired on the three circuits so as to show light regardless of which third of the system is in use.

In a central office is installed a switch-board, a motor generator, and certain other machinery. From this office are run two sets of three circuits, each with a common neutral for each set, to furnish current for the lamps of half of the structure, and two similar sets of three circuits each to furnish current for the lamps of the other half. The two outside wires of each circuit, together with its series wires, are run in one metal conduit, and the neutral for each set of three circuits is run on glass insulators.

The conduits and wires are supported on short wooden cross-arms fastened on the top of each floor-beam between the two outermost stringers on each side. They are thus well sheltered, on each side and above, by the concrete floor.

From the office to the structure the four neutrals are combined into two wires, each of solid copper, No. 10, B. & S. gauge. Elsewhere, each neutral consists of a solid copper wire, No. 12, B. & S. gauge. The wires connecting two lamps in series are of the same character, and of No. 14 gauge. Where placed on glass insulators, the neutral wire is of single-braid, Okonite insulation, and all feeders and branches run in metal conduit have double-braid insulation. Joints in the wires

are soldered and double-taped with Okonite and friction tapes. The conduits are of loricated metal, and where branches are taken off, Crouse-Hinds condulets are inserted.

The feeder wires of the lamp are led down through the post to a fuse-box fastened near its bottom, in which is a 6-ampere cartridge fuse. To reach the fuse-boxes from below the floor, a 2-in. pipe was placed in the concrete at each lamp-post, and through this is passed a light ¾-in. metal conduit, bent to prevent the entrance of water, and bushed at each end.

All branch wires running under feeders from the conduit are encased in continuous lengths of Alphaduct. Corresponding with the variation of the size of the wires, the conduits are varied, the six carrying main feeders being 1¼, 1, and ¾ in. in diameter, respectively.

There was serious trouble in entering the several wires into the conduits, because of their comparatively small size; slightly larger conduits would have simplified the task.

The company furnished the lamps, globes, and certain castings, but the contractor installed them, and furnished and installed all other equipment for $28 000. There are about 500 lights on the main viaduct.

6.—Approaches and Laterals.

As has been stated, legal complications prevented the main lateral on Mulberry Street from being built as at first planned, and it became impossible to effect a landing at 12th and Liberty Streets. This lateral was always considered of vital importance to the success of the enterprise as a whole, and when the original plans had to be modified it was decided to build to 12th Street—two blocks east of the former corner—and there install electric elevators of sufficient size and capacity to raise to the deck of the structure any vehicle with its horses. Further delays of a legal nature arose, however, and at present, December 1st, 1908, the Mulberry Street lateral is built to the corner of Mulberry and 9th Streets, and two elevators are installed there. The Hickory Street branch of this lateral divides from the main branch about 300 ft. from the main viaduct, runs down on a 3% grade, turns under the former, crosses Mulberry Street, and ends at 9th and Hickory Streets. This branch is shown on Fig. 2, Plate VIII.

These laterals provide only for the highway loading, the Mulberry Street portion having a roadway 24 ft. wide, and the Hickory Street

branch a roadway 20 ft. wide. The pavement floor, hand-rail, and details are the same as for the main viaduct. The detailing of the steelwork is very similar, but of very much lighter construction; it is a two-column structure with longitudinal girders, transverse beams, and longitudinal stringers. No towers or longitudinal bracing are used; the columns are fastened down with four anchor-bolts which provide the necessary stiffness in every direction.

The foundations are principally of small concrete pedestals resting on sand. They are placed at a sufficient depth below the street grade to allow for cellar construction. Two of the elevators have been installed. Each has a counterweighted moving platform, 11 by 32 ft., lifting a total distance of 40 ft. 9 in. They are designed to carry 22 000 lb. at 50 ft. per min., or 11 000 lb. at 100 ft. per min. One 50-h.p. motor is used for each elevator. Automatic gates are provided above and below and on the platforms.

These two approaches were finished and opened to traffic a few months after the completion of the main structure.

7.—Conclusion.

The distances saved by the viaduct are shown by the following computation:

Distance between Sixth and Bluff Streets and Fourth Street and Minnesota Avenue.

By Metropolitan Street Railway line.....	11 950 ft.
By Sixth Street Viaduct.....	8 400 "
Saving.....	3 550 ft.

Distance between Eighth and Wyandotte Streets and Fifth Street and Minnesota Avenue.

By Metropolitan Street Railway line.....	16 000 ft.
By Sixth Street Viaduct.....	11 850 "
Saving	4 150 ft.

The entire construction of the main viaduct was completed in 18 months; in fact, the work was done with such dispatch that several contracts were finished ahead of their allotted times. The substructure was ready for steel in 7 months. The steel erection occupied 10

months, the concrete floor was in construction 3 months, and the asphalt was laid in 1 month.

It is not possible, without lengthy explanatory detail, to give exact distributed costs of the complete structure. These figures have been compiled and compared minutely with the preliminary estimates. When due allowance is made for various changes, additions, and deductions, which originated after the first estimates were prepared, a very close agreement is seen. The greatest variation between actual and estimated cost is to be found in the items "right of way" and "legal expenses", both of which are materially in excess of the estimates.

Contracts were made on the basis of unit-price payments for quantities, and provision was made for all anticipated contingencies. The extras on all contracts were performed at actual cost plus 10 per cent. The total amount of such bills on the two principal contracts was about $\frac{8}{10}$ of 1% of the total final estimates.

The total amounts paid to the contractors, and the total cost to the company, of the principal physical features of the structure are given below, with the unit-prices for the work.

The costs refer to the Main Viaduct, which includes the Kaw River Bridge.

Substructure:		
All pedestals, abutments, earth work, etc.	$283 000	
Three piers in the Kaw River	98 000	
		$381 000
Superstructure:		
Steel erected and painted	$785 000	
Concrete floor	95 000	
Pavement	35 000	
Street-car deck and trolley	86 000	
Lighting system	28 000	
		1 029 000
Right of way		227 000
Total		$1 637 000

Unit-Costs of Work.—

Concrete in pedestals, including excavation, forms, etc.	$10 per cu. yd.
Concrete in shafts of river piers	11 " " "

Mass of foundations of pneumatic piers. (This comprises the gross volume of base, including timber, iron, concrete, and cost of sinking) ..... $18 per cu. yd.
Anchorage metal in place ..... 4¾ cents per lb.
Creosoted piles furnished at site ..... 46.40 cents per lin. ft.
Driving creosoted piles, measured below cut-off ..... 30.25 to 31.35 cents per lin. ft.
Concrete piles in place ..... 88 cents per lin. ft.
Structural metal delivered, erected and painted ..... 3.31 cents per lb.
Construction of concrete floor (cement and bars furnished) ..... $6.085 per cu. yd.
Asphalt pavement ..... 1.05 per sq. yd.
Laying ties for street-car deck (all material furnished) ..... 5.00 per 1 000 ft., B. M.
Laying, bolting, and spiking rails ..... 4 cents per lin. ft. of single track.

Costs per Linear Foot.—

Viaduct with 38-ft. Roadway, and Two Car Tracks.

Substructure	$38.15	per lin. ft.
Steel	92.60	" "
Floor and pavement	18.95	" "
Street-car deck	10.75	" "
Lighting system	3.50	" "
Total cost	$163.95	per lin. ft.

Viaduct with 30-ft. Roadway, and Two Car Tracks.

Substructure	$38.15	per lin. ft.
Steel	88.75	" "
Floor and pavement	15.35	" "
Street-car deck	10.75	" "
Lighting system	3.50	" "
Total cost	$156.50	per lin. ft.

Kaw River Bridge, 30-ft. Roadway and Two Car Tracks.

Substructure	$163.00	per lin. ft.
Steel	198.50	" "
Floor and pavement	15.35	" "
Street-car deck	10.75	" "
Lighting system	3.50	" "
Total cost	$391.10	per lin. ft.

The small difference in cost between the 30-ft. and the 38-ft. roadways is accounted for by the fact that the former is built strong enough in all details to allow for widening it to 38 ft. by lengthening the roadway cantilevers.

Considering both the roadway and the total width of both street-car tracks as upper surface area, the costs per square foot of this area were:

For 38-ft. roadway portion	$2.66	per sq. ft.
For 30-ft. roadway portion	2.93	" "
Average for viaduct proper	2.835	" "
For Kaw River Bridge	7.32	" "

Personnel.—This viaduct has been a work of many minds and many hands, and any list of those associated with it must necessarily be incomplete. The structure was designed by, and constructed under the supervision of, the firm of Waddell and Hedrick, composed of J. A. L. Waddell, and Ira G. Hedrick, Members, Am. Soc. C. E., Consulting Engineers. (This firm has since dissolved.)

Mr. J. H. Thompson, of New York, was Chief Engineer, and had, as his local representative, Mr. E. S. Whitney. V. H. Cochrane, Assoc. M. Am. Soc. C. E., was Engineer in Charge of Shop Inspection. L. R. Ash, Assoc. M. Am. Soc. C. E., was identified throughout with the designing and preparation of the plans. The writer served as Resident Engineer for the Consulting Engineers, and was in general charge of the construction.

The general contract for the substructure was in the hands of Mr. James F. Halpin, of Kansas City, under whom were various subcontractors. The pneumatic piers were built by Kahmann and McMurray, of Kansas City. The concrete piles and most of the timber piles were driven by The Foundation Company, of New York, under

the personal direction of the late George Adgate, M. Am. Soc. C. E. The steel was furnished and erected by the Riter-Conley Manufacturing Company, of Pittsburg, with Mr. John P. Wagner as local representative.

The concrete floor was built by The Expanded Metal and Corrugated Bar Company, of St. Louis, under George R. Heckle, Assoc. M. Am. Soc. C. E. The asphalt paving was laid by the Parker Washington Paving Company, of Kansas City. The laying of the street-car deck, certain substructure, and other various work was done by H. C. Lindsly and Son, of Kansas City. The lighting system was designed by Messrs. Weeks and Kendall, Consulting Engineers, and installed by The Squire Electric Company.

The writer remembers with appreciation the efforts of a number of assistant engineers and inspectors, who labored zealously and efficiently. Grateful thanks are due to many gentlemen for aid in collecting the foregoing data, particularly to Messrs. Waddell, Hedrick, and Cochrane.

DISCUSSION

DANIEL BONTECOU, M. AM. SOC. C. E. (by letter).—This paper presents many points of interest, and is well worth careful reading. Mr. Bontecou.

Since the author gives some details of the preliminary investigation of probable profits, it would be of interest to know more of the assumptions made as to the proportion of the total team traffic which could be depended on to use the viaduct, and of the extent to which they were realized. The estimates of cost and the operating expense given would mean that the gross earnings of the completed structure would need to be about $900 per day. The revenue from street-railway traffic could be determined fairly well, but could hardly be estimated at more than, say, $125 per day, leaving a very large proportion of the earnings to be derived from team traffic. In view of the risk in estimating the use which would be made of a utility of this kind, where it is optional to pay a toll or to take another and slightly less convenient route, it is a fair question whether so expensive a structure should have been built otherwise than by the communities served; or, whether a cheaper structure should not have been built to serve the street-car traffic only, in which case no operating expense except maintenance would be involved.

Since all the cars, and probably most of the other vehicles, must meet grades steeper than 1½%, it would seem that the viaduct roadway might have been placed with advantage nearer the surface, and still have afforded the proper clearances.

The shore line of the Missouri River at the site of the trestle on the accretion was about 600 ft. south of the structure only some twelve years before the latter was built, and has been regulated by protection work a few miles above, and also near the mouth of the Kaw. The design of the trestle foundations, therefore, involved a careful consideration of the permanence of the river regulation. The experience of the 1903 flood seems to the writer to justify the choice of pile foundations, but the conditions of soil and loading appear to call for piles longer than the concrete piles described.

In building the foundations for some large engines in similar ground near the viaduct, tests on a surface of 2 sq. ft. at a depth of about 15 ft. showed a supporting power of 1 340 lb. per sq. ft., and a settlement of 0.3 ft. under a load of 2 800 lb. per sq. ft. It was necessary to use oak piles only, 22 to 24 ft. long, and 300 were driven in an area of 2 100 sq. ft.—the intention being to depend on the combined resistance of the piles and the compressed sand, silt, and clay. After allowing for the frictional resistance of the supported concrete, the load amounted to 36 000 lb. per pile, or 4 480 lb. per sq. ft. of foundation, the value of the piles, by the Wellington formula, being 30 000 lb.

Mr. Bontecou. This foundation was deeply submerged in 1903 without the occurrence of settlement.

In cases where piles are dependent on friction, in somewhat plastic clay, there is probably an element of safety in driving them with a heavy hammer and low fall, rather than jetting them to place, and in arranging them as far as practicable so as to confine and compact the soil. The concrete piles used in the viaduct had a surface area of 130 sq. ft., and, as placed and tested, showed a resisting power of 38 000 lb., or rather less than might be expected from piles driven in a cluster in such material, and probably less than would be desirable if the roadway were actually loaded to 100 lb. per sq. ft.

To determine the effect of clay and dirt on the strength of concrete a good many tests have been made, with varying results; and, although the writer has often permitted an admixture of 5% of earthy material, he has preferred usually to wash dirty stone in barrows having open-mesh bottoms until the water ran clear. It is difficult to understand why the presence of clay should add anything to the strength of concrete, as shown by the author's tests, and if stronger concrete can be made of well-mixed and graded materials with clay, than without it, the writer must confess to a great deal of misdirected energy in the past.

The Kaw River spans, as described, are an interesting example of riveted construction. In view of the high ratio of dead to live load, it would seem to many that a smaller impact allowance was permissible for the motorway load, and the well-understood advantages of pin connections seem to apply to this case with special aptness.

A comparison of the weight and cost of the adopted design with one based on three pin-connected trusses for each span, with cross-beams carried entirely above the top chord would be an interesting addition to the paper.

Ever since the flood of 1903 there has naturally been a feeling of apprehension regarding the Kaw River, and the requirement by the War Department that bridge piers be carried to bed-rock has been cheerfully accepted, regardless of the fact that, while fifteen local bridges were destroyed by that flood, there was no case in which a well-constructed pile foundation failed. It is not apparent, therefore, why it should have been thought necessary to excavate the bed-rock to receive any part of the pier construction.

Mr. Cochrane. VICTOR H. COCHRANE, ASSOC. M. AM. SOC. C. E. (by letter).—The writer left the office of the Consulting Engineers to take charge of the shop inspection. In some respects the work was unique, and a brief account of it, in addition to what the author has given in his unusually complete paper, may be of interest.

There were three inspectors on the work, two employed by the Consulting Engineers, and one by the manufacturers of the paint specified for the structure. While in the office, the writer, in the

capacity of Chief Draftsman, had assisted in designing the main portion of the viaduct, and was familiar with the locality where the structure was to be built; consequently, he was well acquainted with the theoretical and practical considerations affecting the design. Accordingly, he was given authority to approve all shop drawings. As the author remarks, much time was saved by this arrangement, the drawings being put in the shops, when desired, almost immediately after being turned over to the writer for approval. The chief saving in time, however, resulted from the advice and assistance the writer was able to give to the force engaged in preparing the shop drawings. It was necessary in several instances to make considerable modifications in the design, and these changes were made without having to await the approval of the Consulting Engineers. The writer's experience in this case convinces him that the inspector in charge of a piece of work of this magnitude should be quite familiar with the design, in both its theoretical and practical aspects. Mr. Cochrane.

The squad of draftsmen assigned for the shop detailing seemed to consist largely of men inexperienced in making drawings for such a structure; and, although the engineers' plans were quite complete, the first shop drawings finished and checked were found to have so many errors that they had to be practically re-checked before being approved. After the men became more experienced in the work, it was not necessary to check the drawings so much in detail. Altogether, there were more than 600 sheets. From time to time, after a number of drawings had been approved, the weight of each member was computed and entered in a quadrille-ruled notebook.

The paint used on the work was guaranteed by the manufacturers for a period of ten years, provided they were allowed to employ inspectors to watch the work in all stages. Their representative in the shops succeeded in getting excellent work, although there was much friction at first. The paint used in assembling was so heavy and stiff that there was considerable difficulty in applying it. The painting was done under contract by Greek laborers, and, at first, it was difficult to get them to clean the metal as thoroughly as desired, since the standard adopted was much in advance of anything to which they had been accustomed; the quality of work required, however, was clearly and forcefully stated in the specifications, and, in cases of dispute, the paint inspector was nearly always sustained by the representative of the engineers.

The punching, assembling, and reaming was carefully watched throughout. Each member after being finished—but before being painted—was inspected for loose rivets and other defects, and all leading dimensions and open-hole spacings were carefully verified. In some kinds of members many pieces were nearly alike. In such cases lists were prepared and used instead of the unwieldy drawings. The

Mr. Cochrane. dates of acceptance were noted on these lists, or on the prints in case they were used. Many pieces had to be sent back for correction, as was to be expected, but no very serious errors were found, and little material was rejected on account of surface defects. The provision for field reaming greatly facilitated the shopwork, but increased the labor of checking up field connections, as no assembling was done, and consequently there was no way to insure the matching of rivet holes except by measuring each connection.

In cases of errors the inspectors endeavored to remedy the defect in such a way as to cause the least possible expense to the shops without impairing the strength of the piece. When the shop employees learned that the inspectors could be depended on to assist them in correcting mistakes, little or no attempt was made to conceal them, and they were usually brought to the attention of the inspector as soon as discovered.

Each day the inspector entered in a scratch-book, without regard to order, the marks and number of pieces of all material inspected during the day. As soon as the invoices of shipments were received, the material listed on the invoices was checked off the scratch-book; at the same time the scale weights were compared with the calculated weights. By this means many mistakes were discovered in the invoices; some were due to misreading the marks painted on the pieces; in many cases, the scales were incorrectly read; and, in some instances, the number of members shipped was in error. The scratch-book was not intended as a permanent record, but each Saturday afternoon a list was made of unfinished work then in the shops, a column headed "Inspected" being filled out from the scratch-book. Unfinished pieces remaining in the shops from week to week were not re-entered in this book unless advanced in construction, in which case they were checked off in a column headed "Remarks." Thus the book showed at a glance the exact state of unfinished work at any time.

A record of shipments was made from the corrected invoices once each week in a book, a column headed "Number Previously Shipped" giving the numbers taken from a previous record-book. Owing to the errors in the invoices, previously mentioned, the shop's records of shipments, after a time, became very unreliable, and it was necessary to depend on the inspector's records altogether.

In preparing the record-books mentioned, the headings of the various columns were not repeated on each page, but were placed near the beginning and end of the book, the tops of the intervening leaves being cut away.

The difference between the estimated and scale weights for the main viaduct was only three-tenths of 1 per cent.

Mr. Mogensen. O. E. MOGENSEN, M. AM. SOC. C. E.—The author deserves credit for describing so fully the construction and design of the Sixth Street Viaduct; it is particularly interesting to know the problems and

difficulties which arose during the construction, and how they were overcome. Mr. Mogensen.

One of the points brought out in the paper relates to the trouble experienced in securing clean crushed stone for the concrete, and the fact that a comparative test showed that the unwashed clayey stone made stronger concrete than the clean washed stone. Mr. Bontecou takes exception to this last statement, which reminds the speaker of tests conducted some years ago by W. A. Rogers, M. Am. Soc. C. E., with a view of ascertaining whether cement with varying proportions of loam had a detrimental effect on concrete, and, if so, to what extent.

With Mr. Rogers' permission, the results of these tests are given in Table 1.

TABLE 1.—TENSILE TESTS OF 1-IN. BRIQUETTES OF PORTLAND CEMENT WITH VARYING PROPORTIONS OF LOAM.

Percentage of loam.	PROPORTIONS:		TENSILE STRENGTH, IN POUNDS PER SQUARE INCH.			
	Cement.	Sand.	28 Days.	3 Months.	6 Months.	1 Year.
0	1	2	503	523	546	563
2			439	504	509	577
5			356	445	454	549
10			346	465	464	566
20			344	368	418	522
0	1	3	292	361	384	430
2			257	390	418	458
5			248	315	352	491
10			242	335	379	506
20			299	334	368	515
Neat			825	841	712	....

The proportions were determined by volume; the sand was obtained by screening gravel from Hammond through a No. 4 sieve; the sand was washed, 0.021 foreign matter being eliminated; the briquettes were kept 24 hours in air, and were then immersed in water until broken.

It will be noted that there is actually a tendency toward an increase of strength for the briquettes after six months and one year; these were mixed in proportions of 1 part cement to 3 parts sand.

In giving these results, it is not by any means the object to encourage an admixture of clay or loam with concrete, but the records of various tests made from time to time show that the engineer is not justified in condemning rock or sand off-hand if a small amount of clay or loam be present. E. C. Clarke, M. Am. Soc. C. E., reports that 10% of loam, used with clean sand and Rosendale cement, did not decrease the strength after six months or one year. He found that 350 briquettes of Portland cement were not weakened by clay in moderate quantities, nor were their weathering qualities impaired in 2½ years.* Nor is the engineer justified in compelling the con-

* *Transactions*, Am. Soc. C. E., Vol. XIV. Additional information may also be found in *Engineering News* (August 10th, 1905, p. 140) in an article by W. C. Hoad, Assoc. M. Am. Soc. C. E., regarding tests made at the University of Kansas, on the effect of clay and loam on the strength of Portland cement mortars.

Mr. Mogensen.

tractor to go to the extra expense of cleaning the stone, if a limited amount of clay or loam is present, particularly if a batch mixer is used. Sticky clay adhering to stone cannot be removed effectively by playing a hose on the stone in a barrel. It is far more effective to run the clayey stone in a batch mixer long enough, and to make use of the grinding effect in removing the clay from the surface of the stone.

Another point which has been brought out is the fact that built-up driven piles were better adapted for the wet soil encountered in the work than the contemplated patented pile. It would be interesting to know why the use of the patented pile was undesirable and inexpedient, even if the ground were wet. The moulding of a concrete pile in place without subjecting it to the impact of the hammer, is unquestionably superior to driving it or sinking it by jetting, especially when the jetting has a tendency to reduce permanently the carrying capacity of the pile by loosening the surrounding ground.

No mention has been made of stations. To an outsider not familiar with the local conditions, it seems that a few stations on the viaduct near the steam railroad stations, would have added, not only to the utility of the viaduct, but also to the earning capacity of the electric railroad and viaduct.

Mr. Blackburn.

N. T. Blackburn, Jun. Am. Soc. C. E. (by letter).—This project seems to have been carefully planned and well thought out, from every point of view, and the paper presents it very clearly.

Evidently, both the concrete and pine piles, under the pedestals, were sunk to full depth by jetting alone, no blows being struck by the hammer. Jetting is used almost exclusively in sinking piles in Galveston, Tex., and all along the Gulf Coast, where the soil to be penetrated is sand. Under that portion of the Galveston sea-wall constructed by the Government, the piles were driven by a steam hammer, the jet being used only through the sand. It was specified that the last 2 ft. which entered the underlying clay should be driven by the hammer alone. The piles were of green pine with the bark on, and of such length that they penetrated the sand and went about 2 ft. into the clayey subsoil, their length averaging about 33 ft. The average number of piles driven per 8-hr. day was 62, a No. 2 Vulcan steam hammer being used. However, there was probably less shifting of the driver than was required for the pedestals of the Sixth Street Viaduct.

At Aransas Pass, Tex., during 1908-09, piles for the trestle used in building the jetty were driven without difficulty to a penetration of 15 ft. by steam hammer alone, the hammer striking at the rate of about 80 blows per minute, and without a jet. The material penetrated was the hard sand of the seashore.

The author's results of tests made to determine the effect of a

small amount of clay in the broken stone for the concrete, seems to bear out what the writer has been able to gather from the experiments of others. Mr. Blackburn.

GEORGE H. PEGRAM, M. AM. SOC. C. E.—This paper is very interesting as describing a modern structure designed to meet unusual conditions. It brings to mind a structure designed by the speaker, and built, in 1886, in Ninth Street—a parallel street. This was the first elevated railroad built outside of New York City. The design is described in a discussion of Dr. Waddell's paper on elevated railroads.* Mr. Pegram.

The structure was designed for steam-train traffic, and the steel-work weighed 500 lb. per ft., or less than half the lightest New York railroad. The distinguishing characteristic was the support of the track rails in the chords of the trusses which were composed of two channels forming a trough, the channels acting as guard-rails for the wheels.

The trusses were pin-connected, and the chords were connected by angle-iron diagonal lateral bracing. No wooden cross-ties were used. The objects were to reduce the noise to a minimum and avoid darkening the street under the structure, both of which were accomplished.

The history of such structures is interesting. Happening to be in Kansas City about twelve years ago, the speaker observed that wooden cross-ties had been placed on the chords, the reason given being that street cars, with small wheel flanges, had replaced the steam trains, and the cross-ties made a safer floor in case of derailment. The speaker was recently informed that the pin-connected truss bracing has been replaced by riveted bracing, which is in line with experience in such construction.

The foundations were the ordinary concrete piers, resting on piles.

E. E. HOWARD, ASSOC. M. AM. SOC. C. E. (by letter).—The detailed data of traffic and income of the viaduct, requested by Mr. Bontecou, are unfortunately not available. In general, the writer's information is that the inter-city highway traffic has been slowly increasing, since the structure was opened, and begins to approximate the estimated volume. Five-sixths of the total highway revenue was expected to come from the laterals into the West Bottoms, and, as explained, these have not yet been built in accordance with the original plan; so that the present structure can hardly be fairly considered in connection with the prospective income of the original estimates. Only two street-car companies are now operating over the structure, but, from the present indications of increasing traffic, the revenue from this source will in time materially exceed the estimated amount. Mr. Howard.

Referring to Mr. Bontecou's suggestion that the structure could have been placed lower, it may be noted that 425 ft. from the east end

* *Transactions*, Am. Soc. C. E., Vol. XXXVII, p. 438.

Mr. Howard. it is down to the clearance required for surface railway tracks, and is at the required height above flood water at the Kaw River. No very great saving would have resulted by increasing the grades, and although, as he remarks, there are grades much in excess of those on the viaduct immediately at each end, the reason for the light grades may be readily found in the original conception of providing a long uninterrupted roadway over which rapid speed could easily be made. His suggestion that it would have been wiser to have built to provide only for street cars finds an answer, as far as the traveling public is concerned, in the flood of 1908 when for a time the viaduct afforded the only means for the passage of vehicles from Kansas City, Mo., to Kansas City, Kans. With respect to the excavation of bed-rock in the Kaw River, it might be explained that the bed-rock was found to be on a decided slope, necessitating leveling; and, as the surface was somewhat shaly, it was thought best to prepare the ditch, as described, around the entire perimeter of the caisson.

The figures given by Mr. Mogensen with regard to clay in stone for concrete are of particular interest. The writer regrets that he cannot give similar detailed figures of experiments with the various materials used, instead of the general conclusions from a few tests, but extensive experiments could hardly have been undertaken at the time. The general observations lead to a concurrence in Mr. Mogensen's conclusion, that probably better results are obtained by more vigorous and extra turning of clayey stone in the concrete mixer, than by washing the stone in wheel-barrows with a hose, as suggested by Mr. Bontecou. It would seem that if stone is dirty enough to demand washing it should be washed by some more certain method, as, for instance, revolving it in a drum under a water-jet.

While it may be possible in one experimental trial to pour enough water into a wheel-barrow to wash the contained stone entirely clean, still, as work goes forward, and with the carelessness of laborers ordinarily employed, such washing on a large scale for an entire job would hardly be of the same thoroughness or of sufficient thoroughness to give good results; or, in other words, if such washing is sufficient, none is probably needed.

The reason for abandoning the patented pile was that sample piles, driven in similar soil, when subsequently examined, were found to be discontinuous. Apparently, the soft soil had pushed in, as the shell of the pile was withdrawn, and occupied some of the space supposed to be filled by concrete. That is to say, the ground had pinched off the pile, so that when excavated it was seen to be merely discontinuous chunks of concrete, which, of course, immediately fell over.

Mr. Mogensen states:

"The moulding of a concrete pile in place without subjecting it to the impact of the hammer, is unquestionably superior to driving it or

sinking it by jetting, especially when the jetting has a tendency to reduce permanently the carrying capacity of the pile by loosening the surrounding ground." Mr. Howard.

This is subject to question. As piles, or, in the case of moulded-in-place concrete piles, forms or cylinders for the piles, are driven rapidly in succession, the ground is very much compressed. This is a familiar phenomenon, seen always in driving piles. It would be of much interest to know what experience has shown to be the effect of such compression on moulded piles which are still green. If the concrete is placed immediately after the moulds are driven, the material would be subject to the compression and vibration of the ground as successive piles were driven, about as the concrete would be taking its set. This reference, of course, is not to a single pile driven by laboratory methods, but as ordinary work would go forward. It seems that the advantages of a made-up pile are very definite, in that the pile can be closely examined just before sinking, so that it can be reasonably well known what is left in the ground.

The latter part of the statement, regarding the reduction of carrying capacity of a jetted pile because of the loosening of the surrounding ground by jetting, may also be questioned. The method given in the paper for approximating the bearing capacity of jetted piles was the only one that the engineers could evolve. If there is any more accurate method, a description would be much appreciated. Manifestly, it is not possible to put a test load on every pile driven for a structure, and it would be interesting to know how the data were arrived at concerning the reduction of the carrying capacity of jetted piles. All engineers who have driven piles in swampy ground have had experience with cases where the pile drives very easily, and, yet, when left to set, can hardly be started again by the same hammer that drove it. Does not a similar action occur in the case of jetted piles, and is not the ground which settles back around the pile at least as solid as in its original condition, and probably more solid and dense? If this is not true, why is it that a hole is often left around the perimeter of a jetted pile where the ground has sunk away? The writer would very much appreciate any data concerning the bearing capacity of jetted piles or the method of determining it.

Replying to Mr. Mogensen's inquiry regarding stations, it should be explained that the conditions at present existing make it unnecessary to have intermediate street-railway stations, as there is practically no pedestrian traffic below the structure, it being paralleled on one side by railroad yards, and on the other side by unoccupied river bottom.

Mr. Pegram's comparison of this structure with the Ninth Street Viaduct, built on a parallel street, is pertinent; and an examination and comparison of the two structures on the ground is interesting, as illustrative of the increase of live loads to be provided for and the

Mr. Howard. general change in details of construction. The pin-connected trusses of that structure were replaced by small riveted trusses in 1905, but the old top chords, each consisting of two channels, are retained, and now act as stringers; thus the columns, the cross-bracing, and the top chords of the original structure are still in use, and are operated over by a frequent and heavy modern street-car service.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1121

SOME EXTENSIVE RAILROAD SURVEYS, AND THEIR COST PER MILE.*

By W. S. McFetridge, M. Am. Soc. C. E.

With Discussion by Messrs. Clinton S. Bissell, F. Lavis, E. W. Lewis, George L. Dillman, and W. S. McFetridge.

The following paper is not a theoretical discussion of railroad location. It is intended to give a general description of some extensive railroad surveys, a brief outline of methods and results, and the cost per mile. The writer does not remember having seen a similar statement covering so many miles, viz., 1 400 miles of preliminary lines and 600 miles of location; and therefore trusts the paper may prove of value. The surveys were completed some time ago, at which time the tables of mileage and cost were prepared, but not heretofore published.

Introductory.—Early in 1902 the Little Kanawha Syndicate began surveys for the extension of its lines, eastward from Palestine, W. Va., to Belington, and westward from Parkersburg, W. Va., to Zanesville, Ohio. About one and one-half years later it also took up the location and construction of a line running northward from Belington to the Pennsylvania-West Virginia State line.

The surveys were conducted under the following charters: Zanesville, Marietta and Parkersburg Railroad, in Ohio; Parkersburg Bridge and Terminal Railroad, from the Ohio-West Virginia State line to

* Presented at the meeting of May 19th, 1909

Parkersburg (this division included a bridge over the Ohio River a few miles below Parkersburg); Little Kanawha Railroad, from Parkersburg to Burnsville; Burnsville and Eastern Railroad, from Burnsville to Belington; Buckhannon and Northern Railroad from Belington to the Pennsylvania-West Virginia State line; in all, some 328 miles of main-line location, exclusive of branch lines.

Fig. 1 (Ohio) and Plate IX (West Virginia) show the country traversed and the main survey lines, many of the short lines not being shown. The lines shown give the general layout. The termini, as usual, were fixed; physical conditions also fixed the Little Kanawha River as the only outlet to the Ohio. These points decided in a general way the proposed route. Owing to local conditions, it was also believed that the heavier traffic would be west-bound, and therefore that every effort should be made to get as low a ruling grade as possible for this traffic.

The desired results may be briefly stated as follows:

1.—Easiest grades possible, especially against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line;

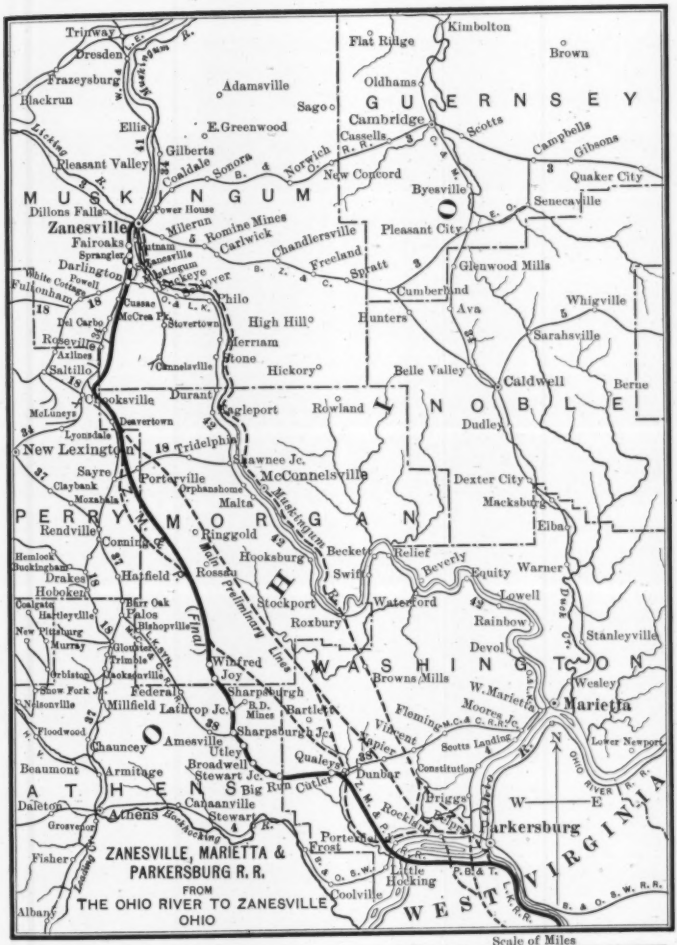
4.—Occupy to as great an extent as possible critical and strategical points, so as to block the route to other lines;

5.—Reach certain definite places previously determined on;

6.—And, taking into consideration the naturally heavy and expensive work on any line, the total cost should compare favorably with the cost of any other line, many miles longer, with heavier grades and curves, but avoiding some of the most expensive work.

All roads previously built through the adjoining regions have long stretches of 1.5% grades, and curves up to 12 and 14 degrees. The first surveys, therefore, were of a preliminary nature, in order to determine what grades and curves could be secured.

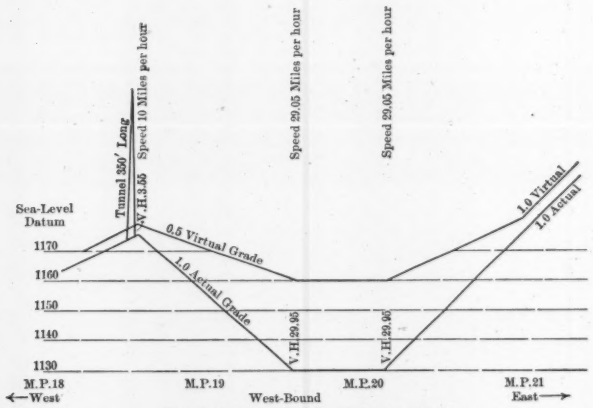
After a number of surveys, locations, and explorations had been made, it was found that the following grades and curves were possible; in Ohio, 0.5% grades, 4° maximum curve; Little Kanawha Division, 0.3% grades, 8° maximum curve; Burnsville and Eastern Division, 1.0% grades against east-bound and 0.5% grades against west-bound traffic, 8° maximum curves; all grades compensated for curvature at the rate of 0.04′ per degree.



3. Baltimore & Ohio R. R.
4. " " " S.W. R.R.
5. Bellaire, Zanesville & Cincinnati Ry.
18. Columbus, Sandusky & Hocking Ry.
34. Cincinnati & Muskingum Valley Ry.
37. Toledo & Ohio Central Ry.
38. Marietta, Columbus & Cleveland R. R.
41. Wheeling & Lake Erie Ry.
42. Ohio & Little Kanawha R. R.

FIG. 1.

These results were kept in view in continuing the surveys, and were obtained in each case. It was desired to avoid all momentum grades, and only in one case was it found necessary to use them. This case occurred at Mile 20 on the Burnsville and Eastern Division, where such a grade was introduced in order to avoid a long détour combined with some exceptionally heavy work. At some future time this grade can be taken out if desired. In the meantime, for any ordinary reason of operation, a train need never stop there, and thus get stalled; there is no station stop, and a water-station or siding is not feasible, on account of local conditions, so that trains can always "run for the hill." Fig. 2 is a profile at this place.

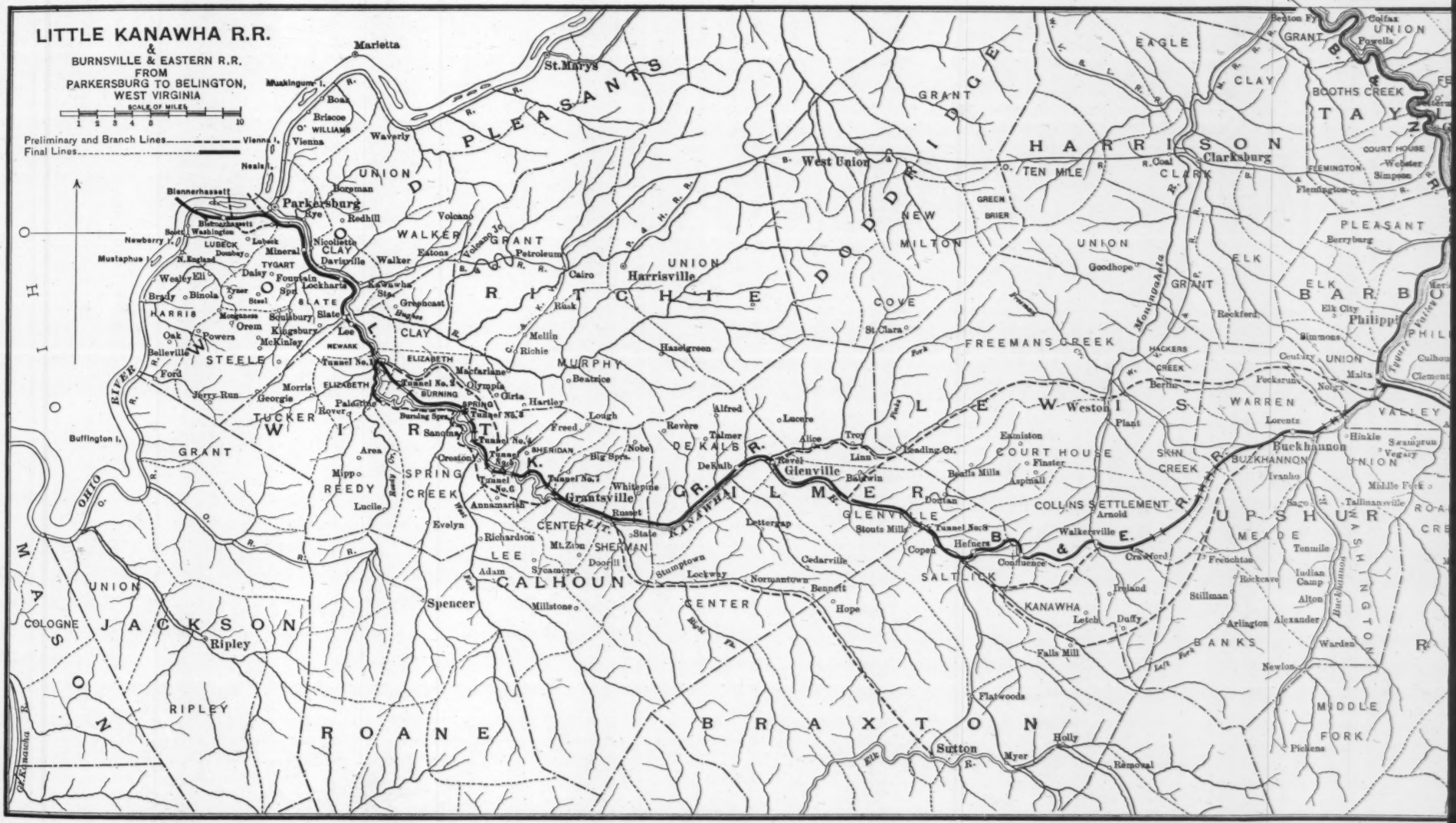


Train loaded for 0.5% grade and able to maintain uniform speed of 10 miles per hour.
BURNSVILLE & EASTERN R.R.
Profile at M.P. 20, showing Virtual Profile for West-Bound Traffic.

FIG. 2.

The grades desired were very easy for parts of the country, and required some rather long continuous grade lines, the longest being on the Burnsville and Eastern Division, where there are 1.0% grades, 7 miles and 7½ miles long, respectively, and a 0.6% grade 14 miles long, all against east-bound traffic. Fig. 3 shows a grade-line profile of this division.

To obtain these grades it was necessary to make a thorough study of the country and a detailed examination of several routes.


LITTLE KANAWHA R.R.
&
BURNSVILLE & EASTERN R.R.
FROM
PARKERSBURG TO BELINGTON,
WEST VIRGINIA
SCALE OF MILES
1 2 3 4 5 10
Preliminary and Branch Lines
Final Lines
To Pa.W.Va.State Line
Marietta
St.Marys
Parkersburg
Belleville
Ripley
Spencer
Sutton
Clarksburg
West Union
Weston
Glenville
Grantsville
Buckhannon
Philippi
Elkins
Harrisville
Ritchie
Doddridge
Pleasants
Wood
Jackson
Roane
Calhoun
Braxton
Gilmer
Lewis
Barbour
Upshur
Randolph
Tucker
Taylor
Harrison
Wirt
Wetzel
Tyler
Ohio River
Kanawha R.
Little Kanawha R.
Hughes R.
West Fork R.
Elk R.
Buckhannon R.
Tygart Valley R.
Monongahela R.
Newark
Elizabeth
Macfarlane
Olympia
Girta
Hartley
Freed
Lough
Revere
Talmer
Alfred
Lacore
Alice
Troy
Linn
Leading Cr.
Esmiston
Finster
Skin Creek
Court House
Buckhannon
Union
Middle Fork
Tallmansville
Sago
Ivanho
Meade
Tennile
Frenchton
Rockcave
Indian Camp
Alton
Stillman
Arlington
Alexander
Warden
Newlon
Middle Fork
Pickens
Holly
Myer
Removal
Flatwoods
Falls Mill
Banks
Left Fork
Ireland
Duffy
Letch
Kanawha
Hope
Bennett
Normantown
Cedarville
Lockwy
Stumptown
Center
Millstone
Doorill
Sycamore
Adam
Lee
Richardson
Mt.Zion
Sherman
Evelyn
Lucile
Annamariah
Creston
Tunnel No.1
Tunnel No.2
Tunnel No.3
Tunnel No.4
Tunnel No.5
Tunnel No.6
Burning Spring
Sheridan
Whitepine
Russet
State
Nobe
Big Spring
Dekalb
Revel
Baldwin
Beulla Mills
Aspinall
Collins Settlement
Arnold
Walkersville
Copen
Hefners
Confluence
Crawford
Doctan
Stouts Mills
Glenville
Lettergrap
Cedarville
Salt Lick
Kanawha
Reedy
Spring Creek
Grant
Area
Mipp
Volcano
Petroleum
Cairo
Harrisville
Union
Walker
Volcano Jc.
Greencast
Slate
Lockharts
Fountain
Daisy
Tygart
Dorsey
Mineral
Lubeck
Washington
Bloennerhassett
Newberry
Mustaphus I.
Buffington I.
Muskingum I.
Neals I.
Vienna I.
Vienna
Boaz
Briscoe
Williams
Waverly
Borgman
Rye
Redhill
Nicollette
Davisville
Eatons
Kanawha Sta.
Rush
Mellin
Richie
Beatrice
Haselgreen
St.Clara
Goodhope
Berryburg
Elk City
Simmons
Century
Malta
Fecksrun
Nolan
Berlin
Hackers Creek
Rockford
Freemans Creek
Cove
Green Brier
Ten Mile
R.Coal
Booths Creek
Court House
Webster
Simpson
Flemington
Colfax
Powells
Grant
Eagle
Clay
Union
Pleasant
Elk
Barbour
Philippi
Warren
Lorentz
Weston
Plant
Valley
Hinkle
Swamptown
Vesuvius
Union
Middle Fork
Tallmansville
Sago
Ivanho
Meade
Tennile
Frenchton
Rockcave
Indian Camp
Alton
Stillman
Arlington
Alexander
Warden
Newlon
Middle Fork
Pickens
Holly
Myer
Removal
Flatwoods
Falls Mill
Banks
Left Fork
Ireland
Duffy
Letch
Kanawha
Hope
Bennett
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Lockwy
Stumptown
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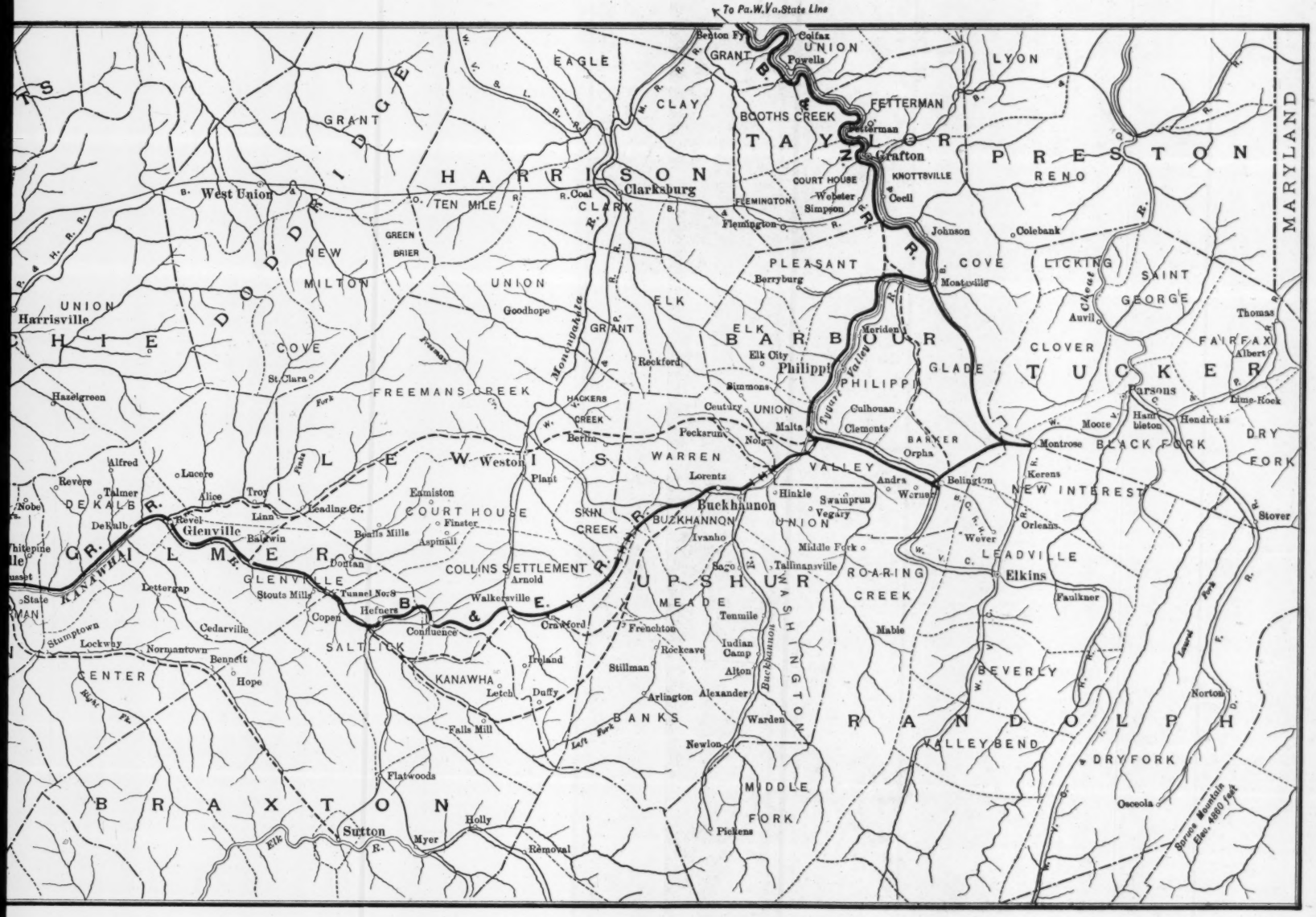
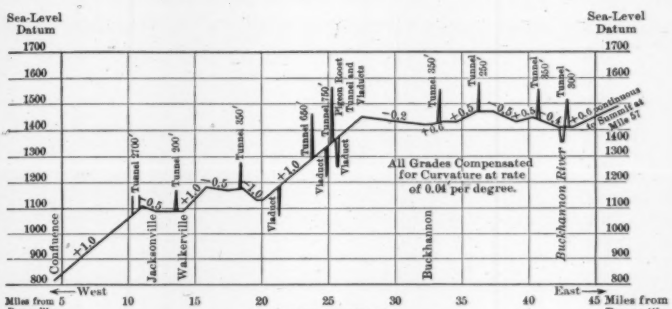
Millstone
Doorill
Sycamore
Adam
Lee
Richardson
Mt.Zion
Sherman
Evelyn
Lucile
Annamariah
Creston
Tunnel No.1
Tunnel No.2
Tunnel No.3
Tunnel No.4
Tunnel No.5
Tunnel No.6
Burning Spring
Sheridan
Whitepine
Russet
State
Nobe
Big Spring
Dekalb
Revel
Baldwin
Beulla Mills
Aspinall
Collins Settlement
Arnold
Walkersville
Copen
Hefners
Confluence
Crawford
Doctan
Stouts Mills
Glenville
Lettergrap
Cedarville
Salt Lick
Kanawha
Reedy
Spring Creek
Grant
Area
Mipp
Volcano
Petroleum
Cairo
Harrisville
Union
Walker
Volcano Jc.
Greencast
Slate
Lockharts
Fountain
Daisy
Tygart
Dorsey
Mineral
Lubeck
Washington
Bloennerhassett
Newberry
Mustaphus I.
Buffington I.
Muskingum I.
Neals I.
Vienna I.
Vienna
Boaz
Briscoe
Williams
Waverly
Borgman
Rye
Redhill
Nicollette
Davisville
Eatons
Kanawha Sta.
Rush
Mellin
Richie
Beatrice
Haselgreen
St.Clara
Goodhope
Berryburg
Elk City
Simmons
Century
Malta
Fecksrun
Nolan
Berlin
Hackers Creek
Rockford
Freemans Creek
Cove
Green Brier
Ten Mile
R.Coal
Booths Creek
Court House
Webster
Simpson
Flemington
Colfax
Powells
Grant
Eagle
Clay
Union
Pleasant
Elk
Barbour
Philippi
Warren
Lorentz
Weston
Plant
Valley
Hinkle
Swamptown
Vesuvius

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1121.
McFETRIDGE ON
RAILROAD SURVEYS.



The topographical sheets of the United States Geological Survey were found of great value in making a broad, general study of the country. At that time the published sheets covered only a small portion of the lines, but, fortunately, this was the central part of West Virginia, where they were of particular value. It may be pertinent to state that these sheets were usually found quite accurate in regard to main summits, large rivers, and towns, but intermediate details were always more or less inaccurate.



GRADE PROFILE,
BURNSVILLE & EASTERN R.R.,
M.P. 5 TO M.P. 45.

FIG. 3.

A large number of maps of small scale (1 in. to 1 mile or even smaller) were compiled and traced from various State, county, and road maps, on which the several survey lines could be indicated. Such maps are of particular value, and should be the first ones prepared—if possible before any surveys are made.

General Layout of Surveys.—The general direction of the survey, except along the Little Kanawha Division, was almost directly across the general drainage of the country.

In Ohio a direct line between termini was first examined, but was found to be impracticable. This line marked the north and east limits of the country to be examined; certain conditions prevented the location of a line north of it. A systematic examination toward the southwest was then made, and finally a satisfactory line was developed, over which a final location was made. All the streams here lie in deep, narrow valleys, and are exceptionally crooked. The only feasible

way to traverse much of the country was to get up out of the valleys and stay out. Such a method necessitated crossing about 100 ft. above several streams, and running short tunnels between the water-sheds; it also gave the shortest line, the easiest grades, and the lightest curvature. Any attempt to avoid the high crossings or tunnels involved long détours, excessive curvature, and heavier grades. On this division more miles of preliminary lines were run, in proportion to the location, than on any other.

The main problem on the Parkersburg Bridge and Terminal Railroad was the determination of the location for a bridge over the Ohio River. The Government regulations required 90 ft. clear head-room above low water and no piers in the main channel, which necessitated a 700-ft. span. Several feasible locations were surveyed, and complete data were obtained at each place. The location finally adopted is about 5 miles below Parkersburg, and is believed to be the shortest and cheapest railroad bridge crossing the Ohio between Pittsburg and the Mississippi, the 700-ft. span practically clearing the entire channel, the eastern approach being on the only bottom lands in that region which are not subject to overflow, and the western end being against a steep bank rising rapidly from the river. The bridge and viaduct approaches are but little more than half as long as the present bridge at Parkersburg.

The Little Kanawha Division, in general, followed the Little Kanawha River. The hills rise abruptly from the river banks, the slopes often being as steep as 1½ to 1. The river is very crooked, and to follow it gave a long line with much curvature. Much distance could be saved by cutting through the country at various points, but the work was very heavy. The bottom lands are narrow and subject to overflow several times a year, the river sometimes rising 20 ft. in a day. A river line would have to be built along the steep side-hills for most of the way (this being expensive), or else be subject to inundation during high-water seasons. The hills are cut through in all directions by numerous tributary streams. The grade of the streams is from 50 to 80 ft. per mile, their length varying from 1 or 2 to 20 or 30 miles, and all of them terminate against abrupt hills. Many previous surveys of this river had been made, and practically all of them followed the river, abandoning any serious attempt to shorten the line materially by cutting across country. The first Little Kanawha

survey also followed the river, and a complete location was made. This line was run principally for information, and, shortly afterward, was abandoned. The main problem on this division was to locate such a line that its total cost would not greatly exceed the cost of a river line equally well built (though no definite amount was ever fixed for this), that its grades and curves would be as light as by river, and that it would be as short as possible. This problem required the most minute study of the land, a thorough knowledge of its character, the running of many lines, and the comparison of many estimates. On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river for any material distance because the cost would be prohibitive; but when a thorough study began to show the length of line that could be saved by a mile or two of enormous work, the question had a different aspect. The only way to get sufficiently accurate data, on which to base final conclusions, was to make the actual surveys of the different routes.

One item which was always more or less an unknown quantity, but which was the deciding point between two lines, in more than one case, was the treacherous character of some of the side-hills. In a number of places (the same trouble was also encountered on other parts of the lines) the side-hills were very likely to slide. It often happened that quite a large area, and sometimes from 15 to 20 ft. deep, would move bodily down hill for several feet when disturbed by any construction work. To avoid these places as much as possible, and to estimate their probable cost when comparing lines, were difficult problems. On location, a plan sometimes used was as follows: Where the grade was near the foot of a hill, the location was thrown away from the hill, in order to avoid cutting the slopes; this gave a fill where it was not apparently necessary, but it was cheaper in the end. When the grade was high up on the hillside, the opposite was done, for the purpose of getting the roadbed on solid ground. In many cases, however, such places could not be avoided in any way, and it was often found that ground, to all appearances perfectly firm, would slide after construction started. In comparing the various lines, this tendency to slide had to be taken into account, but the estimate of cost was of necessity more or less an approximation. These slides, throughout this country, are frequently a very large item of cost in building, and are usually underestimated.

The line, as finally located, is a combination of river and cross-

country line. It is 31 miles shorter than the river, in a total distance of 100 miles. There are eight tunnels, usually short, the longest being 4 000 ft. There are seven river crossings, with main spans from 100 to 300 ft.

The Burnsville and Eastern Division is in the central mountain part of the State. The highest altitude reached is 1 725 ft. above sea level. At the junction with the Little Kanawha line, the altitude is 750 ft. The lines generally run east; the mountains or hills, north and south; the drainage, generally north. There were several intermediate summits to be crossed. It was about as difficult to find the 0.5% grade west as to find the 1.0% grade east. The first line examined was up the head-waters of the Little Kanawha, up Fall Run, and through the head-waters of French Creek. The country was very rough and broken, and supporting ground for grades could not be found. This line also developed the fact that, owing to the rapid rise of the land at the eastern end, it would be necessary to use the maximum grade at once for getting out of the Little Kanawha water-shed, and then follow around the head-waters of the streams to the north, in order to get supporting ground. The line finally located was outside of the Little Kanawha water-shed.

The Buckhannon and Northern Division followed the river for about two-thirds of its length, the other third being cross-country. The general problems of location were very similar to those on the Little Kanawha and Burnsville and Eastern Divisions.

Methods Used.—Field parties were made up as follows:

	Monthly Salary.
Assistant Engineer in charge	$125 to $150
Transitman	85 " 100
Levelman	75
Rodman	65
Head chainman	50
Rear chainman	45
Rear flagman	40
Stakeman	35
Axemen (from two to five)	30
Topographer	65
Tapemen (two)	45
Draftsman (part time)	60

Camp outfits were not used. The parties boarded at houses along the line. This was often a disadvantage, on account of difficulty in getting quarters, especially for a full corps; but, on the other hand, the party could frequently make its headquarters at some town and drive to and from the work, so that probably this method served just as well as furnishing camp outfits.

Each party was given from 40 to 60 miles of line to cover, depending on local conditions.

The following abstracts from the "General Instructions" give an idea of the work required to be done by each party:

"The Assistant Engineer will receive instructions as to when and where preliminary lines will be run, grades proposed to be used, what the line is intended to develop, and the general information necessary in regard to the same. He will then be responsible for the amount and character of the work done by his party, and for the proper development of the country over which he works.

"Topography will be taken on lines as instructed."

(Topography was taken on practically all lines except in Ohio.)

"After preliminaries are run, the Assistant Engineer will be instructed over which line to locate.

"All location will be first projected on topography sheets, etc.

"No location will be assumed as final until examined and revised, if necessary, by the Assistant Chief Engineer, and also approved by the Chief Engineer."

The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence. On location and in particularly difficult country he was to be with the party almost constantly.

After the first route to be examined had been chosen, a preliminary line was run through; then the alternate routes were run, all surveys being tied together; and finally the lines required for a thorough development of all possible routes were run. Although the first preliminary line might give the grades and curves desired, and possibly be the line over which the final location was made, it was not finally determined on until the subject was fully investigated.

Sometimes all the preliminary data required over a certain route may be obtained quickly by stadia methods, but some sort of a definite

line is required from which reliable conclusions can be drawn. Especially is this true in heavily timbered, hilly country, where the range of vision is very limited, and traveling on foot is excessively wearisome, and unless such a method is used some feasible route is very likely to be overlooked. Many times on these surveys the Assistant Engineer was sent back over parts of the line to hunt out something better, quite often with success.

In locating long grades, it was preferable to start at a summit and run down hill. With a little experience, the Assistant Engineer could make a sufficiently close estimate of the amount to allow for compensation for curvature, and could run his line accordingly. In the mountainous part of the country here described this compensation amounts to about 6 ft. per mile, equal to 0.12% grade and preliminaries; for a 1.0% compensated grade, run on an 0.88% straight grade, gave the desired information.

In case of the choice of two apparently equal routes, a location was made over each, and estimates were prepared for comparison before choosing a line. In following the larger watercourses, it was usual to locate a line on either side for purposes of comparison, and in order to determine the advisability of crossing from one side to the other either to get a better line or to block the country against rivals.

Some branch lines were located, and quite an extent of line was located and afterward abandoned by reason of change of plans, but at the conclusion the miles of line actually located footed up one-third more (this is practically the correct proportion, but it varied a little more or less on different divisions) than would be required on the entire layout. This extra location was partly for comparison, but largely for reasons mentioned later. In addition to this, many miles of paper location, not located on ground, were made for the same reasons.

It may appear to some that there was much unnecessary location and running of preliminary lines, but in rough country like this, and on work of this magnitude (in 220 miles of this line there were twenty-one tunnels, the longest being 4 000 ft., five viaducts from 400 to 1 000 ft. long, and more than 100 ft. in height, besides numerous other bridges), it is time and money well spent. In no other way can the exact data be gotten, and it leaves no question as to the available routes and the grades obtainable.

The locating engineer, or others on the ground, may feel certain that a line is not feasible, but it is hard to furnish proof, both to himself and to others who are interested, but who will never be over the ground, that such is the case, except by map and profile. Again, an apparently hopeless line may show up much better than expected, and *vice versa*. In this the writer must not be misunderstood to mean that anyone can get the best location if he only runs lines enough; such is far from the case; lines can be run indefinitely without securing the desired results, unless the proper judgment and knowledge are combined with them, the location being usually the most difficult problem of railroad work. What is meant is that it is necessary to run a sufficient number of lines, preliminary and location, to arrive at correct conclusions and to get the requisite exact data to prove the conclusions.

Topography, showing contours, houses, roads, watercourses, etc., etc., was taken on practically all lines. This was taken on 12 by 18-in. sheets. The transit line was plotted each day on the requisite number of sheets; a light pencil line, at right angles to the center line, was drawn through each station, for ease in plotting the topography; elevations were marked at each station; the stations where contours crossed the center line were determined from the profile and marked; connections to other sheets were shown, and then the sheets were ready for field use. The lines were plotted to a scale of 200 ft. to 1 in. The topography was plotted in the field. A hollow drawing-board, 18 by 24 in., was used. The sheet in use was tacked to the board, and the additional sheets were carried inside. A strap around the shoulders of the topographer served to carry the board, and formed a support while plotting (Wellington's method).

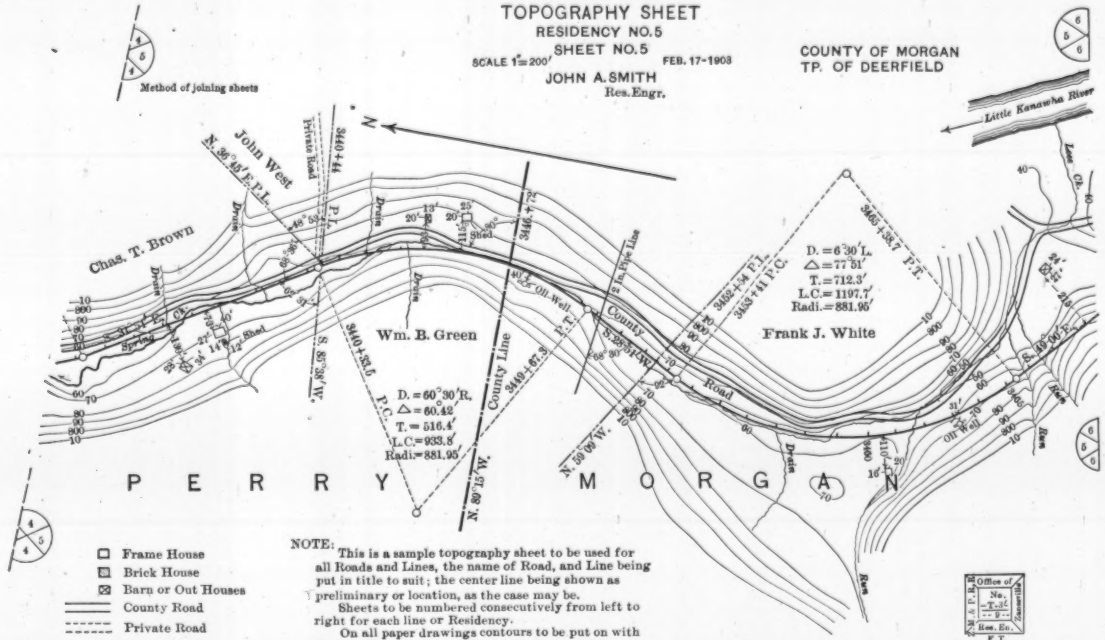
This method was preferred to any other; it is quicker; saves much copying and plotting; the work can be plotted better in the field, where everything can be seen at the time of plotting; and at night the Assistant Engineer has a finished map to look over and study. The topography was taken accurately by using a metallic cloth tape for distances and a hand-level for elevations. Only in this way can one get a projected location to correspond closely with the actual one. The topography was ordinarily taken for 300 ft. on each side of the center line; at particularly difficult summits or similar places a strip from 1 000 to 2 000 ft. wide was often shown, the necessary topography being obtained by auxiliary lines. The sheets were inked in each night.

To obtain a large general map showing all lines, the lines were carefully plotted on tracing cloth, the small sheets were fitted so as to make the center line on each sheet fit the center line on the tracing, and then the topography was traced. By this method any error in plotting or joining the small sheets was eliminated from the large map. This tracing, from which blue prints were made as required, was retained in the office, the sheets being used for rough work and field use.

The location was projected in pencil on sheets in the usual manner, care being taken to determine all angles between tangents by the calculated courses from each sheet, not by using a protractor, or by measuring intersection angles, although either might be used as a check. After the line was located on the ground, the location was accurately tied to the preliminary line and replotted on sheets; the pencil projection was then erased, if so desired.

In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections. The lines always varied more or less in their relative positions. Also, it was often found desirable to change the location at places, giving a corresponding change in the profile; for instance, it might be decided in the field that, owing to surface conditions, certain hills could be hit harder or perhaps avoided. The most frequent change was to put the line harder into steep side-hills. When projecting lines along such places, there is nearly always a tendency to fit the line too closely to the surface and not to allow enough cutting to put the roadbed on firm ground. Owing to these facts, it is usually better to have the first location projected and run in by the field engineer, who is most familiar with actual conditions; after that the revisions and necessary changes can be taken up by the higher officers. Projections made by anyone not thoroughly familiar with the ground should be used with caution. The best location cannot be obtained without the topography and a projected line; also, it cannot be obtained on the sheets alone, without a thorough and complete knowledge of the character of the country; the two must be studied together, the field engineer at all times corresponding freely and fully with the Chief Engineer and getting his ideas and instructions.

After the first location had been made, it was studied further in



NOTE: This is a sample topography sheet to be used for all Roads and Lines, the name of Road, and Line being put in title to suit; the center line being shown as preliminary or location, as the case may be.

Sheets to be numbered consecutively from left to right for each line or Residency.

On all paper drawings contours to be put on with brown ink.

FIG. 4.

the Chief Engineer's office; if any changes were desired they were taken up with the Assistant Engineer, usually by the Assistant Chief Engineer and the Assistant Engineer going over the ground together and there studying the question. After settlement, the location was taken as final for that route, and the maps and plans were brought up to date.

Standards.—All curves of 3° or more had spiral approaches. These were allowed for in cross-sectioning by offsetting the slope stakes the required distance. The spiral was in standard form, so that, when the degree of curvature was known, the offset from the simple curve and the length of the spiral were obtained from tables. For simplicity and ease, all records, profiles, etc., were kept on simple curve data. When spiral curves came in tunnels, a special plan was made for each case, showing the offsets from the tangent and the simple curve at every 10 ft. on the spiral, the alignment being kept on the tangent and the simple curve, and allowing the required offset in giving the widths for the tunnels. All grades were compensated 0.04′ per degree of curvature.

Vertical curves were inserted at all places when the change of grade was more than 0.1 ft. in 100 ft., a standard form being used in which the length of the vertical curve varied directly with the change of grade.

The usual weekly reports, maps, and profiles were sent in by the Assistant Engineers. Standard forms were used for all notes, maps, profiles, plans, and reports. The standard rules were first issued as typewritten copies, and standard forms were adopted from time to time as needed; later, they were compiled and issued in book form, being blue-printed from tracings, making 22 pages of general rules and information, together with 24 standard forms for notes and plans. It was found to save much work and time in the Chief Engineer's office to have all data in uniform shape; much correspondence was likewise avoided, as employees could readily find sizes, scales, etc., for plans, and also other information. Therefore they were not compelled to write to headquarters, and thus avoided the delay of two or three days required for letters to go and come.

A few of the standards used extensively on location are here shown.

Fig. 4 shows the standard topography sheet, the same form being used on preliminary and construction lines. After the line was finally determined, sheets were prepared showing that line only, and the title was made to suit the construction work shown.

The standard titles for profiles are shown by Fig. 5, and the standard titles for field books, by Fig. 6. The standard forms for transit and level notes are shown by Figs. 7 and 8, respectively. Fig. 9 shows the standard form for a location profile, and Fig. 10 shows the standard form for a situation survey for a bridge. When first prepared the structure was shown in pencil, and was not inked in until the detailed

B. & N. R. R.

Profile of { "A" Preliminary / Projected Location. / "L" Location / Revised Location.

from ____ to ____

Sta. ____ to Sta. ____

Date

____ Chief Eng'r. Name of Engineer.

B. & N. R. R.

Profile of Revised Location

on

Subdivision No. ____ Div. No. ____

Sta. ____ to Sta. ____

Date

____ Chief Eng'r. ____ Resident Eng'r.

B. & N. R. R.

Progress Profile.

Between Sta. ____ & Sta. ____

Divn No. ____ Subdivn No. ____

Sect'n ____ to ____ inc.

Date

____ Chief Eng'r. ____ Resident Eng'r.

Note – Titles to be put in upper left hand corner of profiles.

FIG. 5.

design had been made by the Consulting Bridge Engineers, A. P. Boller and H. W. Hodge, Members, Am. Soc. C. E. The standard form of report of openings required is shown by Fig. 11.

Cost.—The greatest number of miles of preliminary line run in one day by one party was 7, and of location, 4½. The location averaged slightly more than 1 mile per day per party, except on the Burnsville and Eastern and on the Buckhannon and Northern lines, where it

L. K. R. R.

Transit or Level } Notes on Prel. line "A" from Grafton to Moatsville, Sta. 0 to 750.

Date_____ Asst Engr or Levelman.

Book No._____

L. K. R. R.

Transit or Level } notes on "L" Location via "A" Prel. from Grafton to Moatsville, Sta. 0 to Sta. 748

Date_____ Asst Engr or Levelman.

Book No._____

L. K. R. R.

Transit or Level } notes on revised location on Subdiv. No._____ Div. No._____ Sta._____ to Sta._____

Date_____

Book No._____ Resident Engr

FIG. 6.

TRANSIT

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	deg Curve	Deflection.	Delta.	Calculated Bearing.	Magnetic Bearing.
401					
+50					
400				S 79°08' E	S 78°45' E.
+53 P.T.		7° 06'			
399		6° 26'			
+50		5° 48½'			
398		5° 11'			
+50		4° 33½'			
397		3° 56'			
+50		3° 18½'			
396		2° 41'			
+50		2° 03½'			
395		1° 26			
+50		0° 48½'			
394		0° 11'			
+85 PCC.	2° 30' L.	3° 00'	14° 12'		
+50		2° 39'			
393		2° 09'			
+50		1° 39'			
392		1° 09'			
+50		0° 39'			
391		0° 09'			
+85 P.C.	2° L.		6° 00'		
+50					
390				S 58° 56' E	S 58° 35' E.

Transit Book

Field books to be paged and indexed.

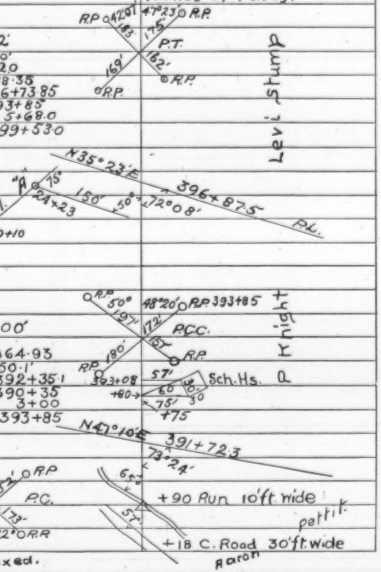


FIG. 7.

Level

Location Name or Letter of Line from (Town) to (Town) Sta. to Sta.

Stations.	Back Sight.	Height Inst.	Foresight.	Elevation.	Grade.	Cut or Fill.	
B.M.				100.00		B.M. No 10. on OAK. 20ft. Left sta. 910+50	Date.
Inst.	10.04	110.04					Names of Party.
910			5.2	104.8	97.00	c. 7.8 +35	H.W.= L.W.=
+40 Run			12.4	97.6	97.20	c. 0.4	+45 Run 10ft. wide.
+90 Road			3.3	106.7	% 97.45	c. 9.2	+74 Road 30ft.
911			2.5	107.5	+ 5/10 97.5	c. 10.0	11+05
T.P.			11.94	98.10			
Inst	0.72	98.82					
912			0.8	98.0	98.0	Grade.	
+50 P.C. 5L			1.2	97.6	98.15	F. 0.6	
913			5.3	93.5	98.30	F. 4.8	
+50			8.5	90.3	% 98.45	F. 8.2	
914			4.6	94.2	3/10 98.60	F. 4.4	
+50 P.T.			3.6	95.2	98.75	F. 3.6	
915							

FIG. 8.

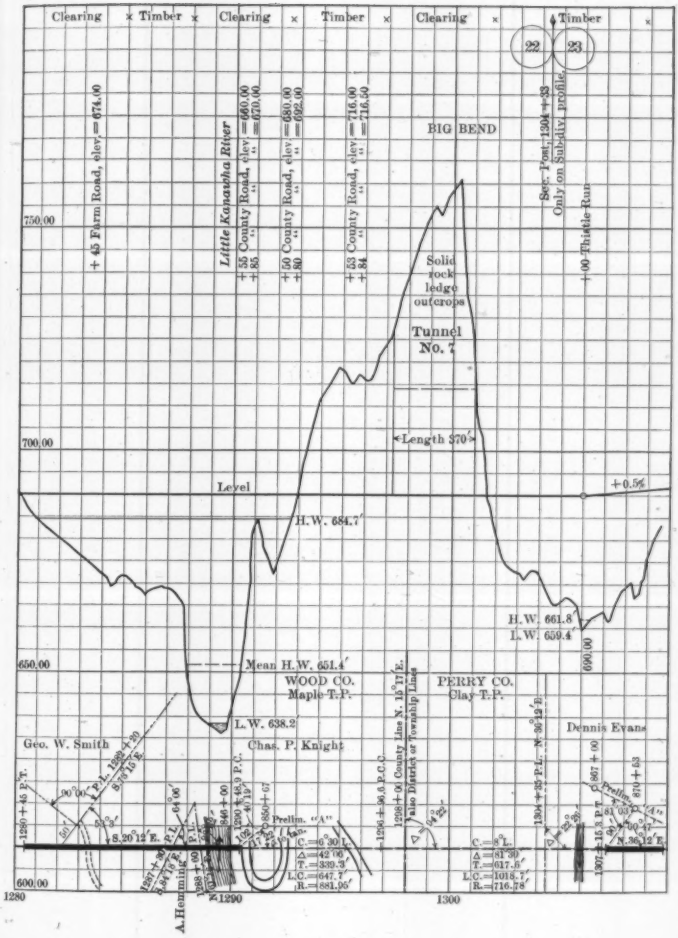

Clearing
Timber
Clearing
Timber
Clearing
Timber
22
23
+ 45 Farm Road, elev. = 674.00
Little Kanawha River
+ 55 County Road, elev. = 660.00
+ 85 " = 670.00
+ 50 County Road, elev. = 680.00
+ 80 " = 692.00
+ 53 County Road, elev. = 716.00
+ 84 " = 716.50
BIG BEND
Sec. Post, 1304 + 33
Only on Subdiv. profile
+ 00 Thistle Run
750.00
Solid rock ledge outcrops
Tunnel No. 7
Length 370'
700.00
Level
+0.5%
H.W. 684.7'
H.W. 661.8'
L.W. 659.4
650.00
690.00
Mean H.W. 651.4'
WOOD CO.
Maple T.P.
PERRY CO.
Clay T.P.
L.W. 638.2
Geo. W. Smith
Chas. P. Knight
Dennis Evans
Prelim. "A"
600.00
1280
1290
1300
A. Hemming

FIG. 9.

averaged ¾ mile. Stakes were set every 100 ft. on tangents, and every 50 ft. on curves. Special pains were taken with the instrument work and measurements, in order to avoid the chance of serious errors in the center line after construction commenced. The speed of location parties was usually limited by the amount of clearing that could be done, but the number of curves and the rough character of the ground were also large factors in limiting the speed.

Each party cost from $35 to $40 per day, being allowed all expenses in addition to salaries.

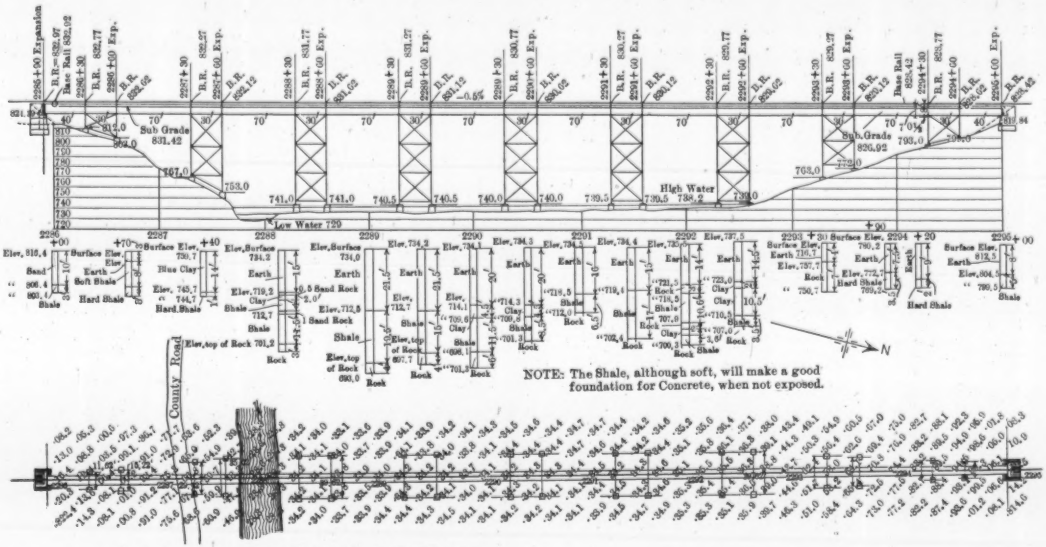
Table 1 gives the cost per mile of the completed surveys. It is to be noted that this is the total cost, and includes office rent, purchase of instruments and supplies, general expenses, all salaries, field expenses, and the preparation of final maps, plans, profiles, and estimates, with everything in readiness to make contracts for the line.

TABLE 1.

Company.	Amount spent.	Miles of Surveys:			Average cost per mile.	Average cost per mile of location.
		Preliminary.	Location.	Total.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
L. K. R. R.	$25 076.83	428.19	193.85	622.04	$40.31	$129.36
Z. M. & P.	19 812.77	509.03	105.23	614.26	32.25	188.28
B. & E. R. R.	20 466.68	241.75	113.70	355.45	57.58	180.00
P. B. & T. R. R.	6 651.98	84.56	38.17	122.73	54.20	174.28
B. & N. R. R.	19 249.94	162.51	151.29	313.80	61.34	127.23
Totals	$91 258.20	1 426.04	602.24	2 028.28	$45.00	$151.53

Column 7 gives the cost per mile of actual location, including preliminary lines. Columns 3 and 4 show that there were from 2 to 5 miles of preliminary lines run for each mile of location, except on the Buckhannon and Northern line. Table 1 also includes 302 miles of check levels, the cost being distributed among the various accounts. The data for the Parkersburg Bridge and Terminal line include surveys and soundings for the Ohio River Bridge. The cost per mile includes the topography on practically all lines, except on the Zanesville, Marietta and Parkersburg line, where it was taken only on the located lines.

The cost shown in Table 1, being the total charge against engineering from the inception of the project to the beginning of construction,



ZANESVILLE MARIETTA AND PARKERSBURG R.R.

PLAN AND PROFILE OF

PROPOSED CROSSING OF SUNDAY CREEK

SUB DIV. 3 STATION 2287+ 92

SCALE OF FEET

0 20 40 60 80 100

Chief Eng'r

FIG. 10.

contains a few items which might well be charged to other accounts than location. Instruments purchased could be a credit; some elaborate property surveys and bridge surveys could be charged to construction, but they probably are not large enough to have much effect on the cost per mile. If taken into account, they would reduce the cost. The cost on the Little Kanawha and on the Burnsville and Eastern Division was increased considerably owing to much work being done during a bad winter, when the weather was very unfavorable. The cost on the Parkersburg Bridge and Terminal line was increased by a large amount of property surveying in the city, and by the surveys for the bridge.

The cost on all the West Virginia lines was increased by the immense amount of chopping and clearing necessary. When mountain laurel was encountered, all the axemen that could be worked could not keep a location party moving.

The influence of the weather is a large item in the cost of these surveys; a line run in the middle of winter may easily cost one-quarter more than if run during more favorable weather.

The average cost of one mile of preliminary or location survey, determined from a detailed study of the daily reports of field parties, of office work done, and similar data, is shown by Table 2 (to the nearest dollar), and is believed to be very close to the actual figures.

TABLE 2.

Company.	Average Cost of One Mile:		
	Of preliminary.	Of location.	Of location, including preliminary.
L. K. R. R.	$25	$74	$99
Z. M. & P. R. R.	23	79	102
B. & E. R. R.	35	105	140
B. & N. R. R.	31	94	125

The figures in Table 2 include all expenses, as in Table 1.

Table 1 shows a large variation in the cost of surveys on different divisions, the cost varying from $128 to $188 per mile, with an average of $151. On the assumption that lines located for comparison or similar purposes should be included in the average, one-third should be added to these amounts, as previously noted; the cost would then be as follows:

STANDARD FORM OF REPORT OF OPENINGS REQUIRED.

Z. M. and P. R. R.

List of Proposed Openings on Sub-Division No. 2. Sections 1 to 10 inclusive.

Sta. 1610 to 2138

Station.	Name of Opening.	Acres Drained.	Elev. of High Water.	Elev. of Low Water.	Area Sq. Ft. High Water.	Proposed Opening.	Adopted Opening.	Remarks.
1611+75	Sunday Creek	Not Estimated; probably 60 sq. mi.	738.2	729.0		Viaduct.	Definitely Determined By Higher Officer than Asst or Resident Engr.	See Situation Survey.
1614+15		5		730.0				Ditch to 1611+75
1815+00	Federal Creek	900	740.0	734.0	125	12' Arch		Co. Road has 14' Arch Drainage Area Cleared Stream has rapid fall.
1992+60	Bill Run	300	742.0	739.0	45	D-4X6		Timbered.
2001+15	Dry Run	120	743.0	742.0	20	6X8		Stream Discharges Much Driftwood.

Note.

No absolute formula used to determine size of opening, mostly determined by size of existing openings and judgment of Engineer.

Following formula used as basis (Myer's)

Area of Opening in Sq. Ft. = $C\sqrt{\text{Drainage Area, in Acres.}}$

Max. for C = 4.0± Ordinary Min. C = 1.0

Min. so as not to give less than 1 sq. ft. per 12 Acres; for the small areas.

Where bridges are required, the opening being determined by economical design of structure.

John Jones
Resident Engr
Jox, Ohio.
May 2, 1903.

FIG. 11.

Low	$171 per mile.
High	251 " "
Average	202 " "

Throwing out of the account the mileage of abandoned lines, branch lines, etc., and charging the entire cost to the main line, terminus to terminus, would give $\frac{\$91\,258.20}{328} = \$278.23$ per mile, which would be rather expensive. This, however, is not a fair assumption, and should not be considered, because many miles of lines not needed to determine the main line were located for other reasons and purposes. Therefore, the plan of throwing out only duplications, for comparisons, as shown in the preceding paragraph, gives the correct average cost per mile for the development of the country, including actual comparative locations where needed. It should also be borne in mind that a large proportion of this duplication was necessary, owing to the laws of West Virginia, which require an actual line, located on the ground, and a complete map and profile of that line to be filed with the Secretary of State, and at the county seat, before a railroad company has any rights, of priority or otherwise, to that route or line. This required complete locations for all proposed branch lines, a large number of which were located, and also a complete location over any route for which it was desired to obtain rights. For these reasons, the lines located account for the excess of the mileage over the actual length of the main line.

On the basis of Table 2, it may be assumed that, where the route has been previously determined within such narrow limits that the preliminary and location lines are of equal length, the surveys will cost from $100 to $140 per mile. This is borne out by the results on the Buckhannon and Northern line where the location and preliminary lines were practically equal and the cost was $127 per mile.

These two statements may be combined and put in the following form:

To locate one mile, including an equal length of preliminary lines, cost from $100 to $140; average	$115
To locate one mile, final location, including from two to five times as great a length of preliminary lines, cost from $128 to $188; average	151

To locate one mile, final location, including from two to five times as great a length of preliminary lines, and one-third of a mile of location for comparison, cost from $171 to $251; average..... $202

A tabulation of the mileage of the Buckhannon and Northern line, with reference to the actual length of line to be built, and showing how the results agree with the averages deduced from Table 1, is as follows, the Buckhannon and Northern line being used because the conditions there make it the best average of "all conditions" encountered on the various lines.

Total miles located.....		151.29
Miles of main line contracted for.....	80	
Miles of main line not contracted for.....	4	
Miles of connecting line located, but which may or may not be built, about.....	26	110.00
Making actual miles.....	110	
Leaving duplications, comparisons, etc.....		41.29 miles.

110 miles cost $19 249.94 = $175 per mile.

Results.—The results obtained by studying and surveying the neighboring country, will amply justify the initial cost, and, without doubt, the final results will prove that it is cheaper in the end. A few thousand dollars judiciously spent on surveys will be more than made up for in construction, not only in the actual cost, but in a better line.

That the desired results seem to have been obtained, may be shown briefly as follows:

1.—Easiest grades: 0.5% west, 1.0% east (all the latter in the 25 miles near the eastern end), other roads using 1.5 per cent.

2.—Curvature: 8° maximum, as against 12° on other roads.

3.—Short line: This is seen best on the map. The Little Kanawha line is 31 miles shorter than the river, the only water route. The Ohio line is 16 miles shorter than the existing lines between termini.

4.—Occupation of critical points: Another line, since built, through the Burnsville and Eastern Division (the Burnsville and

Eastern line never having been constructed) and a few miles on Little Kanawha Division, had to locate on and occupy said lines for about 15 miles, although using 1.5% grades and 12° curves. It could not have been built for its present cost without making arrangements with the Burnsville and Eastern and Little Kanawha Railroads.

5.—The definite places previously determined on were, of course, reached.

6.—It being decided, after many surveys—by which the advantage of this line proved so apparent—to build the line as described, no complete estimate of the cheapest possible line in first cost was prepared, but a sufficient number of estimates were made to show that the line as located would certainly fulfill this requirement.

All the work was carried out under S. D. Brady, M. Am. Soc. C. E., then, as now, Chief Engineer of all these companies, who gave much personal attention to the work, and to whom the writer is indebted for assistance and information in preparing this paper, and for permission to publish the maps and costs.

The writer was Assistant Engineer on the Little Kanawha Division, in charge of location; on its completion he became Assistant Chief Engineer of all the companies, both on location and construction.

DISCUSSION

Mr. Bissell.

CLINTON S. BISSELL, M. AM. SOC. C. E. (by letter).—The author states specifically that the paper is not a theoretical discussion of railroad location, and later he cites as the "desired results":

1.—Easiest grades possible, against west-bound traffic;

2.—Lightest curvature;

3.—Shortest line, etc.

He also mentions ruling grades of 0.3% on the Little Kanawha Division, and 0.5% on the Burnsville and Eastern Division. The momentum grade at Mile 20 on the latter division is introduced "to avoid a long détour"; and, in fact, the reader is impressed throughout the paper by the evident great advantage of the "shortest line."

To the writer, a railroad is simply an industrial plant, instituted to furnish transportation, and the "desired results" to be realized, are embodied in the one problem, viz., to obtain the line of greatest efficiency. The author has mentioned no measure of efficiency, nor has he indicated any limitations as to the duty required of the line. Without these, the writer is at a loss to understand the basis upon which the location was made, unless it was merely to get a tolerably direct line, with gradients as low as good direction would allow. The author has either disregarded or withheld one element which Wellington rightly cites* as of "overwhelming importance," namely, the probable amount of traffic. The second important element—that of easy gradients—the author has observed without correlation to the probable traffic. The other two elements—curvature and distance—are, within reasonable limits, minor details of location, and yet apparently the author has given them great attention, to the negligence of one of the two more important elements.

Since nothing can well be further removed from the theoretical side of the problem of location than the actual dollars and cents which the road can either earn or save, let us consider such part of the profile as the author has shown in Fig. 3, between Burnsville and Buckhannon.

From Buckhannon westward it is noticeable that the two stretches of ruling 0.5% grade amount to about 3 miles in 33, or only 10% of the distance, neglecting the momentum grade which, according to the author, can be removed.

Assuming at random that the line will carry annually 3 000 000 tons of freight westward, with heavy locomotives and modern high-efficiency cars, the 0.5% ruling grade permits a train load of 1 700 tons of lading; and the same train, returning with about one-third as much lading, can surmount the 1% grades going east. Taking the

* "The Economic Theory of Railway Location," by the late A. M. Wellington, M. Am. Soc. C. E.

Mr. Bissell.

train-mile cost at $1, for convenience, the annual cost of operation can be roughly estimated.

It is rather surprising that the two ruling 0.5% grades were not made 0.3% to accord with those on the Little Kanawha Division. For the sake of example, it will be assumed that this could have been done at the expense of 4 miles more of distance at a gross cost of $50 000 per mile. For the 0.3% grade the train load going west would be 2 420 tons of lading; and the same train would return east with about one-fifth as much lading, against the 1% grades. Hence, the following figures may be deduced:

3 000 000 ÷ 1 700 × 33 × $1.....west-bound	$58 240	
Same trains returning.....east-bound	58 240	
Annual operating expenses on 0.5% grades.....		$116 480
3 000 000 ÷ 2 420 × 37 × $1.....west-bound	$45 870	
Same trains returning.....east-bound	45 870	
Annual operating expenses on 0.3% grades.....		$91 740
Difference		$24 740
Less interest at 4% on cost of 4 miles of line.....		8 000
Annual saving.....		$16 740

At 4%, this saving justifies a present expenditure of $418 500 to reduce the ruling grades from 0.5% to 0.3%; this reduction increases the efficiency of the line by a saving of 14% in expenses; and probably there would be enough capital left to take out the momentum grade at Mile 20.

Unless the matter of location be viewed from some such standpoint as this, the writer does not understand how the "shortest line" practicable on the "easiest grades possible" and on the "lighest curvature" can be decided. The intimate correlation of these three elements does not permit of such a decision, except upon some basis other than simply the contour of the country.

A few years ago the writer made several surveys and locations along the south shore of Nova Scotia between Cape Sable and Shelburne. The drainage, flowing down to the sea, crossed the direction of survey about at right angles. The route along the seashore was too irregular to justify a low line for any considerable distance, and a high line several miles inland touched but few towns, because these, for the most part, were situated along the shore. The final finished location was a compromise between these two lines. The whole route was wooded, especially on the slopes of the ridges where the maximum grades occurred. The specifications of the Dominion and the Provincial

Governments fixed the maximum grade at 1.5% and the maximum curvature at 6 degrees. Mr. Bissell.

In conducting these surveys, the writer found it necessary to take contours only on the ruling grades and a few other points on the line. The grades were first cut through, and the profile taken, after the manner mentioned in this paper, from the summit downward. To accomplish this object, the transit was frequently carried several miles ahead of the measured line and set on the summit or other governing point, and the profile run from a temporary bench-mark, the elevation of which was assumed or estimated with a barometer. The measured line was then brought forward, connected, and run again over the maximum grade. Particular attention was then given to the contours and to the topography of the stream crossing which was usually found at the foot of the grade. The great advantage in this method is the fixing of the proper point of connection at the foot of the grade with the assurance of passing through the governing point ahead. The writer believes that notwithstanding the necessity of running over the grade twice, time is saved by avoiding much backing up and subsequent alteration of the line back of the foot of the grade. It gives a continuous chainage, saves much contouring, and produces a preliminary map which lies close to the location at all the critical points.

The author's figures on costs are very interesting and valuable; but the maximum rate of progress mentioned, 7 miles of preliminary line and 4½ miles of location per day, has never been equalled by the writer. He considers it excellent work for one party to accomplish even half of these distances in one day.

F. Lavis, M. Am. Soc. C. E.—This paper is a valuable addition to the somewhat scanty literature relating to the subject, and, undoubtedly, will be of considerable interest to those connected in any way with the conduct of surveys for the location of railroads. It is especially so to the speaker, as the methods used agree very closely with those developed by him between 1898 and 1902 on the Choctaw, Oklahoma and Gulf Railroad.* Those surveys resulted in the construction of more than 800 miles of railroad, and, probably, at least as many more miles of location were actually staked out on lines which were not built. The speaker has no exact figures at hand, but it is probable that more than 5 000 miles of preliminary were run during that time. Mr. Lavis.

During the last ten or fifteen years these general methods, with slight minor variations to suit local conditions and individual ideas, have become fairly well recognized as standard on most of the larger railroad systems of the country; at any rate, those engaged to any

* *Transactions*, Am. Soc. C. E., Vol. LIV, p. 104 (1905).

Mr. Lavis. extent in building new lines and extensions, recognize the absolute necessity of such methods, in order to determine without doubt that the line finally adopted is such "that no other line can be built through the same country, with the same or better ruling grades, with less expenditure, at the same unit prices."*

The data relating to costs show close agreement with the costs presented by the speaker in the paper referred to, and, although, in both instances, they are somewhat higher than a great many locating engineers think their surveys have cost, it will generally be found that, when these low costs are given, the fact has been lost sight of, that on any survey for the location of a railroad, the costs, both of the field parties, and a proper proportion of the expenses of the main and division offices, covering all work from the reconnaissance to the completion of the final maps and profiles, ready for the contractors to bid on the construction, and the real estate agents to purchase right of way, should be included.

The surveys described by Mr. McFetridge were made in 1902 and later; the costs previously given by the speaker were for surveys made during the latter half of 1902, so that the time agrees fairly closely. During 1903, of course, it was easier to get men; and salaries, as a rule, were comparatively low, so that the costs, during the latter part of the time when these surveys on the Little Kanawha were made, would tend to be low rather than otherwise; this is shown also by the salary list.

Comparing the costs with those presented by the speaker,† it may be noted that the costs of the field work of the preliminary and location lines agree very closely. The average cost of the lines, as given in Table 2, shows the preliminary to have been about $28, and the location $88 per mile, whereas the costs given by the speaker were $26 and $71, respectively. The costs of the completed location, as described in the paper, vary from $127 to $188 per mile, averaging $151, on the basis of the mileage of located line staked out; or from $171 to $251, averaging $202, if the mileage of adopted location be taken, which latter may be compared with the cost of $192 per mile of adopted location as given by the speaker. The costs of field parties per day are given by Mr. McFetridge as between $35 and $40, whereas those described by the speaker cost from $42 to $65, and averaged $46. The somewhat lower cost per day of the field parties on the Little Kanawha, although the cost per mile of line was higher, is probably due to the difference in organization. On the Choctaw, although all lines were subject to revision, of course, by the Chief Assistant Engineer and the Chief Engineer, the Locating Engineers, whose whole expense, in addition to that of an Assistant Locator, was charged to the field party, were

* "Railroad Location Surveys and Estimates," p. 8.

† *Transactions*, Am. Soc. C. E., Vol. LIV, p. 133.

Mr. Lavis.

expected to be able to decide questions which, on the Little Kanawha Railroad, were apparently not left to be settled by the Assistant Engineer in charge in the field.

Judging from the description of the country and the number and length of preliminary lines run, it would appear that the two surveys are fairly comparable. The country described by Mr. McFetridge is in many cases rougher in detail than that through which the surveys described by the speaker were made, though, in the latter case, the character of the country required the detailed examination of a much wider range in order to determine the best location. This is shown by a comparison of the mileage of preliminary with that of location; on the Little Kanawha lines there were 2.37 miles of preliminary to 1 mile of location, whereas on the lines described by the speaker, the average was 3.14, a little greater than 30% more. The effect of the rugged detail is shown in the considerably increased cost of staking out the located lines on the Little Kanawha.

The generally prevailing idea as to the cost of railroad surveys is, that in the United States it should be about $100 per mile; and in a review* of the speaker's "Railroad Location Surveys and Estimates," a well-known railroad engineer made the statement that "much excellent location has been made in America in not easy country for less than $50 per mile of located line, and this cost includes the needed preliminary for that located line." He adds, referring to the supposed high costs of the Choctaw surveys, "We must conclude * * * that the parties were not made up of veterans." Veterans or not, the speaker does not believe that any line, outside of a tangent across a prairie, and sometimes not even that, can be located and proper maps completed, for any such sum as he mentions, and that for any ordinary country a rough estimate of $200 per mile would be much nearer correct for properly conducted surveys and the preparation of a proper set of plans ready for construction, these latter, of course, not including details of structures. The trouble with most of the published costs of location surveys is that they do not include anything but the cost of staking out a location on the ground; and supervision, office expenses, equipment, costs of preparation of proper maps, profiles, etc., are omitted.

There is considerable difference, also, in the amount and kind of information developed by surveys. As an instance of a not uncommon form presented to a contractor as the location profile, Fig. 12 shows a part of a profile of a line in the Southwest, which was actually sent to a New York contractor, and on which he was asked to make his bid, this bid to be a lump sum for the whole line. Fig. 13 shows a section of a typical profile of located lines such as is usually required by most of the railroads in the West. The material

* *Technical Literature*, April, 1907, p. 175.

Mr. Lavis. in the cuts is explored by making test pits and soundings, as are also the foundations for structures, when necessary, and the information obtained is shown on the profile. The quantities in cuts and fills are given, with the approximate length and location of the haul; the quantities in structures are estimated and shown. In short, the profile contains all the information necessary to prepare a close estimate of cost, and gives the contractor all the information, as to the nature of the work, which he requires in order to make an intelligent bid.

The profile shown by Mr. McFetridge in Fig. 9, as the standard form for a location profile, seems to the speaker to be defective, inasmuch as it shows no quantities, gives little indication of the material to be excavated, or the amount or direction of haul, and no indication of the form or location of bridges, culverts, or other structures, or the quantities of material required to build them. The information required for the proper design of the larger bridges is shown in good shape on the situation plan, Fig. 10, but it is very desirable to show on the profile the type of structure and the space it occupies.

The speaker believes it to be of the greatest importance, not only to show the distribution of quantities on the location profile, but also to insist on their being worked out on the profiles of all projected locations. There is nothing which gives a man as completely the intimate knowledge of the line which it is really necessary for him to have as to work out the distribution, and get a clear idea of what he is to do with the excavation, and where he is to get material for the embankments.

One very important rule in regard to the final location, the speaker believes, should be made inflexible: After the profile of the located line has been platted, a temporary grade line fixed, tentative determinations of bridges, culverts, etc., made, and preliminary quantities calculated and distributed, this latter being most important, the Locating Engineer should take the profile into the field, and, walking over the line, should study carefully the situation on the ground in the light of the information which he then has, which, by that time, should be almost complete. This final review, if conscientiously done by a competent man, will often save many thousands of dollars in the construction, and avoid costly mistakes.

Although, in general, the methods described by Mr. McFetridge correspond closely to those described in the speaker's paper, previously referred to, a few minor differences may be noted. In keeping level notes the speaker prefers to keep the rod readings on the turning points entirely separate from the rod readings for elevations. It seems hardly necessary to have columns for grade and cut-and-fill in the location level notes, these belong more properly in the level notebook of the Resident Engineer, in which he records the check levels run on assuming charge of his residency.

Mr. Lavis.

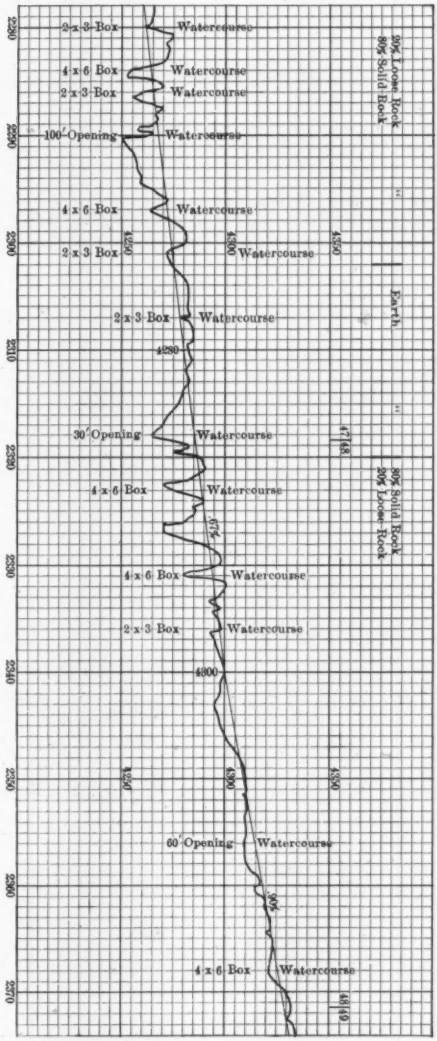


FIG. 12.

Mr. Lavis.

The titles of profiles are generally more convenient when placed on the outside at each end; even if put inside they should be at each end rather than in the upper left-hand corner. Profiles accumulate rapidly on location, and with the title inside and only at one end, the chances are that two-thirds of them will have to be entirely unrolled to find the one sought.

The question of the advantages of separate sheets as compared with rolled maps has been discussed too often for the speaker to hope to say anything new at this time. Mr. McFetridge, however, although he evidently prefers the separate-sheet method, recognizes the necessity of having a large rolled map of the whole line in order to get a comprehensive view of the whole situation. The speaker takes exception to his very positive statement that the separate-sheet method of obtaining topography is quicker and better than any other; to say the least, this is open to argument. The speaker, who has had some little experience, prefers the method described in his paper.

For small general maps of the country, on which—in addition to the principal roads, trails, streams, etc.—may be shown the various preliminary lines as well as the final location, the speaker prefers a uniform scale of 5 000 ft. to the inch. These maps are useful, not only to the field parties and in the office of the Chief Engineer during the location, but also in discussions and conferences with the officials of the road, and others who only require general information. They are also of convenience to contractors in bidding on the work, in order to enable them to form a better idea of its accessibility, etc. They should always be prepared at the beginning of the survey, and information should be added as the survey progresses.

The speaker hardly sees the necessity of staking out a location for the purpose of choosing between two apparently equal routes, or, as was done in this case, for comparing the lines on either side of the larger watercourses. If the preliminary lines have been run with any degree of skill, and the topography has been properly taken, there should be no difficulty in getting, from a projected location on the map, and an inspection of the ground where this projected line would lie, all the information necessary for the comparison of any line or lines, except under the most exceptional circumstances.

The salaries named seem to be somewhat lower than those usually paid. In the West it is not uncommon to have a locating engineer in charge, with an assistant who looks after the details of the field work, leaving the former free to explore the country thoroughly, and devote more attention to the office work and the projection of various locations than he otherwise could. With a conscientious locating engineer this is a very good arrangement, and allows a much more careful threshing out of the country than would otherwise be possible. The author states that, on the surveys described:

Mr. Lavis.

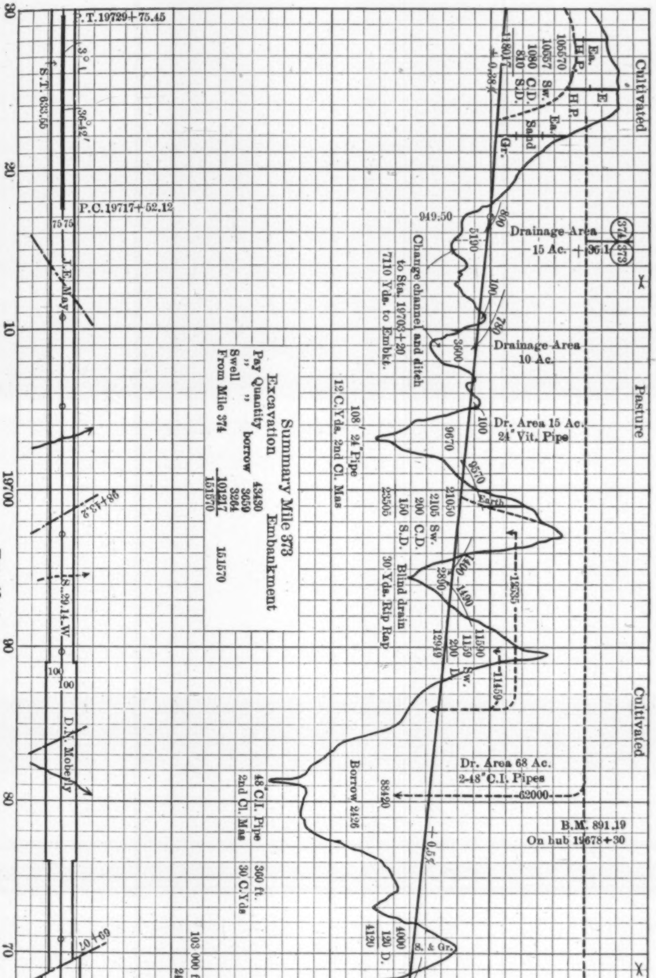


FIG. 18.

Mr. Lavis. "The Assistant Engineer was not expected to spend all his time with the party, but to be with it enough to see that work was going on satisfactorily, and that lines were being run over the proper routes; the remainder of his time was spent in a thorough study of the country, picking out routes to be examined, and looking after his necessary office work and correspondence."

The man who is responsible for the work of the party in the field should be with it most of the time, and if this devolves on the man in charge, it keeps his nose too close to the grindstone to allow of a proper perspective from which to view the problem as a whole, to say nothing of taking up practically his whole time. A proper, careful, study of the many possible projections cannot be made at night by a man who has put in already 10 or 12 hours' hard work in the field, although it is really surprising how much work men on locating parties manage to get through and thrive on.

The salary of the topographer is given at $65 per month, and, without intending in any way to cast any reflections on the young men who filled these positions, this seems to be altogether too little for good men. Taking topography has always seemed to the speaker to be a very simple operation, and it really is, if one has any sort of instinct for it, but it is remarkable how few men seem to grasp the idea, even after considerable experience. The speaker believes the topographer should be paid only slightly less than the transitman.

The fundamental idea of the proper principles governing the staking out of the located line is stated very concisely by the author in the third paragraph on page 116, and this idea cannot be impressed too strongly on all connected with work of this class. "In staking out the location, the aim was to get a profile to correspond with the projection, and not to get the lines in exactly the same relative positions shown by the projections." In other words, there is to be no slavish following of the projected location as a line on the map, but rather to reproduce, on the ground, the line which the studies of the map and the general situation have shown is there, and is the best.

As the speaker has already pointed out elsewhere,* it is just here that modern methods differ from the old. It is neither the old paper location, made without reference to conditions in the field, nor the work of the Genius, described by Mr. Whinery† in his discussion of the speaker's paper, who threw his curves at the hills, as the cowboy throws his rope at a steer, and sometimes hit them and sometimes missed. If he missed, so much the worse for the railroad. The modern method collects all the available data affecting the location, eliminates definitely the unavailable lines, projects on the map the line which best fulfills all the requirements of the situation, and then uses all the

* "Railroad Location Surveys and Estimates," p. 125.

† *Transactions*, Am. Soc. C. E., Vol. LIV, p. 143.

intelligence of the Genius in reproducing that line on the ground. No extraordinary mathematical calculations are required to lay out the located line from the projection, only plain common sense and locating skill. Mr. Lavis.

No amount of accuracy in making the original surveys and platting the map can compensate for lack of intelligence in transferring the projection to the ground, and extreme refinement of accuracy is wasted, if proper intelligence is used.

The statement on page 111, contains, in a very small compass, the essence of the requirements which should govern the conduct of all location surveys; a "most minute study of the land, a thorough knowledge of its character, the running of many lines, the comparison of many estimates," are absolutely necessary in every case. The Locating Engineer, and by that is meant the man really responsible for the location, should be thoroughly steeped in the details of the country, and the word steeped is used advisedly, in the sense that there should be time enough in all cases to allow the information acquired to be taken in, digested, and to permeate him through and through.

One other point brought out in the paper, which is most important, is that actual surveys of all possible routes were made; as the author points out, this does not mean that an indiscriminate topographical map of the whole country was made, or was necessary. In spite of all that has been written about eliminating certain routes by eye, and perhaps by a hand-level, there is no question that absolute proof, by actual survey, should be furnished to the people putting up the money, or, at any rate, to those responsible for its expenditure, that there is no doubt that the location adopted is the best possible under the given governing conditions. The ability and so-called "eye for country," ascribed to the older generation of locating engineers, which many of them undoubtedly possessed, are just as necessary to-day, but must be supplemented by scientific methods, adequate surveys, and much hard work; and this is necessary, not only in order to be able to furnish proof that the line adopted is the best, but generally in order to find this line. As stated also in the paragraph referred to, "On a first examination of this country one was inclined to say offhand that it was impracticable to leave the river * * *; but * * * a thorough study * * * gave the question a different aspect." This experience has been duplicated many, many times, and in many cases the study was not made, and the railroad paid the bill later.

Not many years ago it was not an uncommon thing to hear some of the Geniuses with "eye for country"—and not much of anything else —talk about the foolishness of trying to get a 0.5% line through a 1.0% country. The results obtained by these surveys are only another instance of the possibility and practicability of finding these lines, if

Mr. Lavis. one knows how to look for them. This description, of work actually accomplished, should be borne in mind by all who have to obtain money for such surveys, and, when necessary, should serve to stiffen their backbone to demand all that is necessary to carry out such surveys in an adequate manner.

The author is to be congratulated on the very clear and concise manner in which the information is presented, and his recognition and presentation of the many vital points which differentiate proper methods of making location surveys and getting results from the methods too often adopted, of simply stringing together a lot of tangents and curves over which a train can be run somehow or other, if the stockholders' money holds out long enough and no one else builds a line alongside over which trains can be operated at half the expense.

Mr. Lewis. E. W. LEWIS, M. AM. SOC. C. E.—This excellent paper is of much interest to engineers engaged in railroad location. The methods outlined by the author are very nearly the same as those used by the speaker in making many surveys in widely scattered sections of the country. The make-up of field parties is identical with that used by him on the Northern Pacific Railway, and quite similar to general practice.

The method of plotting topography in the field, while advantageous in many respects, would be a difficult performance in a rainy country, such as the Pacific Northwest, where it rains almost every day for long periods during the winter season.

The speaker prefers taking the notes in a topography field book. These books being ruled like cross-section paper, the contours can be plotted and the exact measured distance from the center line given for each 5- or 10-ft. contour, making it very easy to plot on a large map in the office. This plotting may be done in the evening, but, ordinarily, it is left until the next day. With a draftsman regularly employed to keep up the map, each day's field work may be mapped on the succeeding day.

The method of taking the contours, by measuring from the center line with a tape, and obtaining the difference of elevation with a hand-level, is in accord with the speaker's practice as being the best when an accurate contour map is desired. He has learned by sad experience that the saving of a little time in taking topography by pacing distances, etc., is often the cause of a much greater loss of time when the paper location is run out.

In regard to spiraling curves, if the line is likely to be a final location, particularly if the prospect is good for immediate construction, the speaker prefers to run in the spirals with the location, although, unquestionably, it saves considerable time not to do so.

The costs of surveys, given in the paper, are interesting as showing the wide differences on various lines. The approximate costs of some surveys with which the speaker has been connected in various parts

of the country may be of interest. On the north bank of the Columbia River, in Washington, over very rough, difficult ground, largely on steep slopes, with numerous short tunnels, where, in some instances, it was necessary to lower men down the cliffs with ropes in order to get the line staked, the cost for a stretch of 32 miles was about $325 per mile. In Massachusetts, through an average country, largely wooded, the cost of 20 miles of location was about $150 per mile. In North Dakota, 60 miles of location over a rolling prairie, and crossing one deep, broad valley, cost $110 per mile. In another section of the same State, over rolling prairie, the cost was $80 per mile. These figures are for the total cost of the final location, including all preliminary, trial, and abandoned lines. Mr. Lewis.

GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—The writer is impressed by the absolute lack of value of this paper to the Profession, the uselessness of the conclusions drawn, and the pernicious possibility of these conclusions being applied to some other case of railroad location. No two locations are alike, and hardly any two are comparable in time, cost, or value. Mr. Dillman.

The writer was transitman on two locations which are worth noting as being extremes in his practice: The first was in Cottonwood Cañon, on the Grand River in Colorado, between Dotsero and Glenwood Springs. Afterward the Denver and Rio Grande built its line on the opposite side of the cañon. There was a great deal of clearing, and the topography was such that direct measurement was often impossible, so that a great deal of triangulation for distance was necessary. This location cost more than $600 per mile.

The second location was on the North Platte River, in Nebraska, extending more than 100 miles northwest from North Platte. The country was open, there was no clearing, the tangents averaged about 9 miles, and the curve angles between them were small. This location cost between $17 and $18 per mile.

These locations were each fully staked on the ground. The costs include maps, profiles, and computed estimates, as well as all preliminary work. The parties had about the same number of men and equipment, except that more axe-men were added to the Grand River party. Each party worked efficiently and sometimes for long hours. If there was any difference, the cheaper location was the more carefully made. Tracings of each map were filed with the Interior Department, accompanying applications for rights of way across Government land, and each location was ready for the construction crews.

These locations are probably not extremes; there may have been cheaper locations made; undoubtedly, there have been more expensive ones. What is desired is to bring out the fact that there is no possible connection between the costs of railroad locations; and that a full knowledge of the cost in one case cannot be applied to any other case.

Mr. McFetridge.

W. S. McFetridge, M. Am. Soc. C. E. (by letter).—The writer appreciates the friendly criticism and discussion of his paper, but does not think necessary to add to it, except in reply to two or three questions.

In reference to Mr. Bissell's criticism that no measure of efficiency, probable amount of traffic, etc., were given, these are about the first matters which must be determined, and must be known before the final details or "desired results" of the surveys are decided on by the officials. It was so in this case, and, while a discussion of them might have proved even more interesting than the matters mentioned in the paper, they did not come under its scope, and were intentionally omitted, as were the reasons for adopting such grades, etc.

In regard to Mr. Lavis' criticism of the final profile, the distribution of material, etc., for estimating and for the contractors, was compiled on a profile almost in the form he gives. This profile was made in the Chief Engineer's office, with the assistance of the Locating Engineer. It seemed to work well, but in cases where headquarters could not be easily reached, the method suggested by Mr. Lavis, showing all this information on field profiles, would be better.

Mr. Dillman apparently has made some wrong assumptions in regard to the paper, as the writer knows of no statements contained therein from which he could assume the meaning indicated by his conclusions. No one who knows anything of surveys would expect any two to be identical in method or cost, but the same general principles will apply to the conduct of any up-to-date survey; and there can be no better way of judging what a survey should cost, than to know what a similar one has cost, and what method and organization were used.

The writer does not claim that any iron-clad rules should be followed, and believes that each survey is a separate study by itself. A knowledge of how different surveys have been conducted, and what they have cost, however, should be of value to engineers engaged on them, and likewise to the people who are paying for them, and who often think a survey is a matter of minor cost and importance. Knowing from previous experience that it is sometimes impossible to get enough money to make any kind of a decent survey, the writer hoped to help a little toward bringing about a just appreciation of the needs of such a survey.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1122

COMPUTATION OF STRESSES IN OPEN-WEBBED ARCHES WITHOUT HINGES.

By C. W. Hudson, M. Am. Soc. C. E.

The principles underlying the following method are often used in finding the stresses in certain classes of statically indeterminate structures built of elastic material, but, as far as the writer is aware, they have never been applied to finding the stresses in an open-webbed arch rib without hinges.

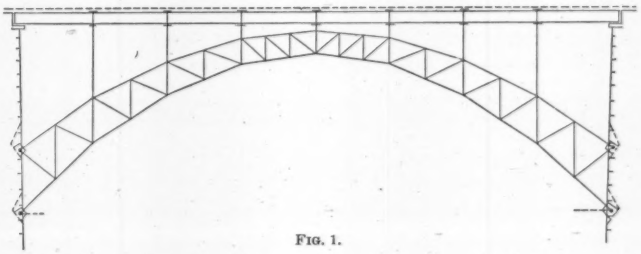


Fig. 1.

If such an arch rib be considered cut, at the crown, by a vertical plane, it is divided into two parts which are each statically determinate.

For the purpose of illustration, assume an arch rib as shown in Fig. 1. Consider the rib cut at the center by a vertical plane; this divides the rib into two parts, as shown in Figs. 2 and 3, the vertical

member at the center being considered divided into two equal parts, one of which is taken with Fig. 2 and the other with Fig. 3.

It is perfectly clear that a load at any point on the left half produces stresses in the truss members of that half which may be determined by statics when each half acts as an independent cantilever truss, and that no stresses can be produced in the right half by this load when the two halves are not connected.

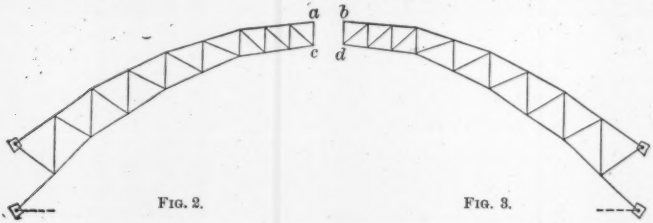


FIG. 2. FIG. 3.

If we suppose the two halves to be connected by joining the points, *a* and *b*, then, under any condition of loading, these two points have the same motion, as they are the same point. Therefore, for a load on the left half it is clear that the stress in any member of that half is that due to the given load when the half is considered as a separate structure modified by the stress acting between *a* and *b* caused by connecting these points, and the stress in any member of the right half is that due to the stress acting between *a* and *b* caused by connecting these points. No matter what the nature and direction of the force produced by joining *a* and *b*, it may be considered replaced by a horizontal and a vertical component.

If we suppose the two halves to be further connected by joining the points, *c* and *d*, then *a* and *b* have the same motion, and *c* and *d* have the same motion under any condition of loading. Therefore, for a load on the left half and the right and left halves connected at *a* and *b* and at *c* and *d*, the stress in any member of the left half is that due to the load on that half, when the half acts as a simple cantilever modified by the forces produced at *a* and *c* by connecting *a* to *b* and *c* to *d*, and the stress produced in any member of the right half is that due to the forces at *b* and *d* caused by connecting *a* to *b* and *c* to *d*. It hardly need be mentioned that the forces, due to connecting *a* to *b* and *c* to *d*, at *a* and *c*, are equal and opposite to those at *b* and *d*, respectively.

No matter what the amount and direction of the forces acting between a and b, and c and d, they may each be replaced by their horizontal and vertical components. For the two halves connected and subjected to any loading, the forces produced by thus joining them may be represented by the four forces of Fig. 4, H_a, V_a, H_c, and V_c, the direction of which may be as shown or the opposite and the magnitude of which is to be determined. Knowing that for elastic structures deflections are proportional to the loads that produce them, four equations between the unknown forces, H_a, V_a, H_c, and V_c, and certain easily determined deflections for the half arch acting independently, may readily be written. The solution of the four simultaneous equations will determine the unknown forces acting at the crown of the arch for each half. These equations of condition for finding the unknown forces at the crown will now be written for a symmetrical arch. The general method is equally applicable to an unsymmetrical arch, however.

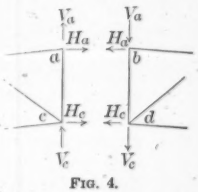


FIG. 4.

Let a load of 1 lb., at any point, x, cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 5.

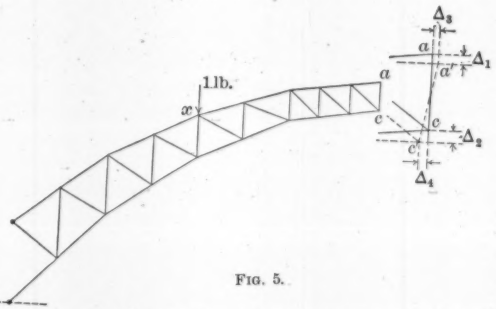


FIG. 5.

Then Δ_1 = vertical deflection of a due to a vertical load of unity at any point, x;

Δ_2 = vertical deflection of c due to a vertical load of unity at any point, x;

Δ_3 = horizontal deflection of a due to a vertical load of unity at any point, x;

Δ_4 = horizontal deflection of c due to a vertical load of unity at any point, x.

Let a vertical load of 1 lb. at a cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 6.

Then d_1 = vertical deflection of a due to a vertical load of unity at a;

d_2 = vertical deflection of c due to a vertical load of unity at a;

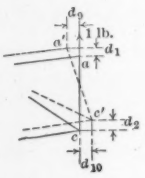


Fig. 6.

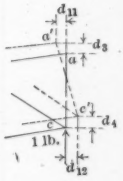


Fig. 7.

d_9 = horizontal deflection of a due to a vertical load of unity at a;

d_{10} = horizontal deflection of c due to a vertical load of unity at a.

Let a vertical load of 1 lb. at c cause the point, a, to take a new position, a', and the point, c, a new position at c', as shown in Fig. 7.

Then d_3 = vertical deflection of a due to a vertical load of unity at c;

d_4 = vertical deflection of c due to a vertical load of unity at c;

d_{11} = horizontal deflection of a due to a vertical load of unity at c;

d_{12} = horizontal deflection of c due to a vertical load of unity at c.

Let a horizontal load of unity at a, as shown in Fig. 8, cause the points, a and c, to take the new positions, a' and c', respectively.

Then d_5 = horizontal deflection of a due to a horizontal load of unity at a;

Fig. 8.

Fig. 9.

d_6 = horizontal deflection of c due to a horizontal load of unity at a;

d_{13} = vertical deflection of a due to a horizontal load of unity at a;

d_{14} = vertical deflection of c due to a horizontal load of unity at a.

Let a horizontal load of unity at c cause the points, a and c, to take the new positions, a' and c', respectively, as shown in Fig. 9.

Then d_7 = horizontal deflection of a due to a horizontal load of unity at c;

d_8 = horizontal deflection of c due to a horizontal load of unity at c;

d_{15} = vertical deflection of a due to a horizontal load of unity at c;

d_{16} = vertical deflection of c due to a horizontal load of unity at c.

The four simultaneous equations will now be written, from which the four unknown forces, V_a, C_c, H_a, and H_c, acting at the crown and due to a load of unity at any point, x, may be determined.

For the points, a and b, and the points, c and d, connected, the motion of a must equal the motion of b, and the motion of c must equal the motion of d.

The downward motion of $a = \Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15}$, and

the downward motion of $b = + V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15}$, and these must be equal, as they are for the same point.

Therefore,

$$\Delta_1 - V_a d_1 - V_c d_3 + H_a d_{13} + H_c d_{15} = V_a d_1 + V_c d_3 + H_a d_{13} + H_c d_{15},$$

and

$$V_a d_1 + V_c d_3 = \frac{\Delta_1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The downward motion of $c = \Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16}$, and

the downward motion of $d = + V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16}$, therefore,

$$\Delta_2 - V_a d_2 - V_c d_4 + H_a d_{14} + H_c d_{16} = V_a d_2 + V_c d_4 + H_a d_{14} + H_c d_{16},$$

and

$$V_a d_2 + V_c d_4 = \frac{\Delta_2}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The motion to the right of $a = \Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7$, and the motion to the right of $b = -V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7$, therefore,

$$\Delta_3 - V_a d_9 - V_c d_{11} + H_a d_5 + H_c d_7 = - V_a d_9 - V_c d_{11} - H_a d_5 - H_c d_7,$$

and

$$H_a d_5 + H_c d_7 = - \frac{\Delta_3}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The motion to the right of $c = -\Delta_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8$,
and the motion to the right of $d = +V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8$, therefore,

$$-\Delta_4 + V_a d_{10} + V_c d_{12} + H_a d_6 + H_c d_8 = V_a d_{10} + V_c d_{12} - H_a d_6 - H_c d_8,$$

and

$$H_a d_6 + H_c d_8 = \frac{\Delta_4}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Solving these four equations, the following values for the unknown forces are found, in terms of twelve easily determined deflections:

$$V_a = +\Delta_1 \frac{d_4}{2(d_1 d_4 - d_2 d_3)} - \Delta_2 \frac{d_3}{2(d_1 d_4 - d_2 d_3)}$$

$$V_c = -\Delta_1 \frac{d_2}{2(d_1 d_4 - d_2 d_3)} + \Delta_2 \frac{d_1}{2(d_1 d_4 - d_2 d_3)}$$

$$H_a = -\Delta_3 \frac{d_8}{2(d_5 d_8 - d_6 d_7)} - \Delta_4 \frac{d_7}{2(d_5 d_8 - d_6 d_7)}$$

$$H_c = +\Delta_3 \frac{d_6}{2(d_5 d_8 - d_6 d_7)} + \Delta_4 \frac{d_5}{2(d_5 d_8 - d_6 d_7)}$$

If the values of Δ_1 to Δ_4, and d_1 to d_8, as found by actual computation, are in the direction heretofore assumed, their numerical value is to be inserted in the foregoing expressions without regard to sign, if they are found to be in a direction opposite to that assumed, they must be inserted with a minus sign.

From Maxwell's Theorem, it is known that:

$$d_2 = d_3, \text{ and } d_6 = d_7.$$

For any particular configuration of members in the arch rib, very much simpler expressions for V_a and V_c may be written.

For the case assumed, to illustrate this paper, a load at any panel point other than *a*,

makes $\Delta_1 = \Delta_2$, and $d_3 = d_4$, and therefore $V_a = 0$.

A load at panel point, *a*,

makes $\Delta_1 = \Delta_2 + (d_1 - d_2)$, and d_3 is still $= d_4$, therefore $V_a = \frac{1}{2}$.

For any panel point of the rib loaded except *a*

$$V_c = \frac{\Delta_2}{2d_4}.$$

And for panel point, *a*, loaded:

$$V_c = 0.$$

Restated, for any panel point loaded other than *a*

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}.$$

And for panel point, *a*, loaded:

$$V_a = \frac{1}{2}, \text{ and } V_c = 0.$$

For such a configuration at the crown as indicated in Fig. 10, that is, where the diagonals adjacent to the crown meet at the top of the crown vertical, $d_1 = d_2$, and d_3 and d_4 are no longer equal.

FIG. 10.

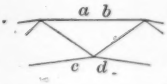


FIG. 11.

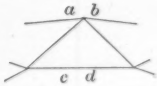


FIG. 12.

Then, for any panel point loaded other than *c*,

$$\Delta_1 = \Delta_2, \text{ and therefore,}$$

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0.$$

And for panel point *c*, loaded, $\Delta_1 = \Delta_2 - (d_4 - d_3)$, and $\Delta_2 = d_4$, and therefore,

$$V_a = 0, \text{ and } V_c = \frac{1}{2}.$$

For such an arrangement of members at the crown as shown in Fig. 11, it is readily observed that

$$V_a = 0, \text{ and } V_c = \frac{\Delta_2}{2d_4}$$

for any panel point loaded.

For such an arrangement at the crown as shown in Fig. 12, it is seen that

$$V_a = \frac{\Delta_1}{2d_1}, \text{ and } V_c = 0$$

for any panel point loaded.

The expressions for the horizontal crown forces remain the same for all the cases selected.

Letting $2\ (d_c d_8 - d_6^2) = n$, and remembering that $d_6 = d_7$, we have, for the value of the horizontal thrusts,

$$H_a = -\Delta_3 \frac{d_8}{n} - \Delta_4 \frac{d_6}{n}, \text{ and}$$

$$H_c = +\Delta_3 \frac{d_6}{n} + \Delta_4 \frac{d_5}{n}.$$

The above expressions for the unknown crown forces have been written from assumed values of the twelve deflections of the half arch acting as a statically determined structure. For any actual problem, these deflections, of course, should be determined accurately, both as to direction and magnitude. The expressions for the value of the unknown crown forces will always be of the same general form as the above, but may differ in some of the signs of the terms.

The resulting plus values for V_a, V_c, H_a, H_c, indicate that they act as assumed, and the minus values that they act in the opposite direction.

Temperature Stresses.—Let Fig. 13 show the motions of a and b, and c and d, for a rise in temperature when each half of the arch is considered as a separate structure, the full lines representing the normal figure and the dotted lines the figure changed by a rise in temperature.

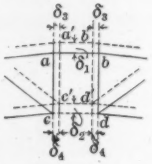


FIG. 13.

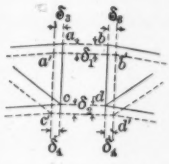


FIG. 14.

Let Fig. 14 show the motions of a and b, and c and d, for a corresponding fall in temperature under the same conception as before.

Then δ_1 = vertical deflection of a and b due to a certain change in temperature;

δ_2 = vertical deflection of c and d due to a certain change in temperature;

δ_3 = horizontal deflection of a and b due to a certain change in temperature;

and δ_4 = horizontal deflection of c and d due to a certain change in temperature.

Four equations for the unknown crown forces due to connecting the two halves may now be written for both a rise and a fall in temperature, using the notation as previously defined.

For a rise in temperature:

The upward motion of $a = +\delta_1 + V_a d_1 + V_c d_3 - H_a d_{13} - H_c d_{15}$, and the upward motion of $b = +\delta_1 - V_a d_1 - V_c d_3 - H_a d_{13} - H_c d_{15}$, for the two halves connected.

These motions must be equal; therefore, $V_a d_1 + V_c d_3 = 0$..... (5)

In an entirely similar manner, the three following equations may be written:

$$V_a d_2 + V_c d_4 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$
$$H_a d_5 + H_c d_7 = -\delta_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$
$$H_a d_6 + H_c d_8 = -\delta_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

For a fall in temperature:

$$V_a d_1 + V_c d_3 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (5a)$$
$$V_a d_2 + V_c d_3 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6a)$$
$$H_a d_5 + H_c d_7 = \delta_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (7a)$$
$$H_a d_6 + H_c d_8 = \delta_4 \quad \ldots\ldots\ldots\ldots\ldots\ldots (8a)$$

An examination of Equations (5), (6), (5*a*), and (6*a*), shows that V_a and V_c are zero; therefore, the only crown forces due to temperature are horizontal.

For a rise in temperature, remembering that $d_6 = d_7$, and making $(d_5\, d_8 - d_6^2) = g$:

$$H_a = -\delta_3 \frac{d_8}{(d_5\, d_8 - d_6^2)} + \delta_4 \frac{d_6}{(d_5\, d_8 - d_6^2)} = -\delta_3 \frac{d_8}{g} + \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = -\delta_3 \frac{d_6}{(d_5\, d_8 - d_6^2)} + \delta_4 \frac{d_5}{(d_5\, d_8 - d_6^2)} = -\delta_3 \frac{d_6}{g} + \delta_4 \frac{d_5}{g}.$$

For a fall in temperature,

$$H_a = +\delta_3 \frac{d_8}{(d_5\, d_8 - d_6^2)} - \delta_4 \frac{d_6}{(d_5\, d_8 - d_6^2)} = +\delta_3 \frac{d_8}{g} - \delta_4 \frac{d_6}{g}, \text{ and}$$

$$H_c = +\delta_3 \frac{d_6}{(d_5\, d_8 - d_6^2)} - \delta_4 \frac{d_5}{(d_5\, d_8 - d_6^2)} = \delta_3 \frac{d_6}{g} - \delta_4 \frac{d_5}{g}.$$

The foregoing gives a simple method for finding the stresses in every member of the arch due to a load at any point, or for any desired change in temperature. Under this method, the work may be easily divided into several sets of independent and easily understood operations at the very beginning of the computations, thereby leaving only the small and easily managed theoretical part for the person in charge.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1123.

HYDRO-ELECTRIC POWER IN CANADA.*

By Cecil B. Smith, M. Am. Soc. C. E.

With Discussion by Messrs. H. Holgate, E. J. Beugler, and Cecil B. Smith.

Introductory.

To view justly the progress which has been made in Canada, in developing water-powers, up to the present time, certain premises must be established, climatic conditions realized, the trend of population, habits, and occupations of the people pointed out, and, above all, the peculiarly favorable topography of the country described.

The population of Canada is familiarly spoken of as agricultural in character, but this is only relatively true, as large and important industries flourish throughout the Eastern Provinces, and, indeed, it is painfully evident that even in this young and sparsely populated country the increase in population is almost entirely urban in character, except on the great wheat plains of the Middle West.

Industries are based on labor, raw materials, and power, and only in the last named would the ultimate conditions be unfavorable to continued progress if coal were the only source of power to be relied on, because Canada is very peculiarly situated as regards its coal areas.

* Presented at the meeting of September 1st, 1909.

Four areas are at present known, as follows:

The Nova Scotian, which is on the extreme Atlantic seaboard, and produces a good bituminous coal, available as far westward as Montreal;

The Souris, which is in Southwestern Manitoba, and produces a low-grade lignite of limited area and value;

The Alberta, which covers a very large area extending from the United States boundary to north of Edmonton and from the plains well up into the Rocky Mountains, the quality ranging from lignite to anthracite, and, as regards both quantity and quality, probably one of the most valuable coal deposits in the world; and

The Vancouver, which is very extensively distributed over the island and up some of the mainland valleys, chiefly developed for export to the Pacific Coast cities of the United States.

Thus it appears that coal is available on the extreme west and east, but for 2 000 miles, covering much of the most populous and fertile center of the country, it is necessary to import coal from the United States for heat and power.

From a hydraulic point of view, however, the country has been peculiarly favored, as a glance at the map will show.

The Hudson Sea is deep set into the heart of the country, and the height of land between the waters flowing to this sea and to the St. Lawrence drainage system is from 1 200 to 1 500 miles long, and, which is of more importance, it is only from 1 000 to 2 000 ft. above the sea, is not of a mountainous character, but is a perfect network of lakes, swamps, rock, and forest, and affords a good reservoir and regulator for hundreds of large rivers, which, as they reach the parent St. Lawrence, descend over precipitous Laurentian rapids, admirable as sources of hydraulic power. This condition obtains all over Canada as far west as Eastern Manitoba, and, as the rainfall averages from 25 to 40 in. per year, with no marked tendency to drought, and as the long winters store up so much moisture that the low-water period often merges into the period of autumn rains, the pinch of low water is not in many cases a serious menace, except in March of severe winters. On the great western plains the topography and climate both militate against hydraulic values, as will be dealt with later.

In British Columbia there are two diverse regions: the eastern and central portions have a low rainfall, and the glaciers feed the

rivers throughout the summer, so that the low-water period is in the late summer and autumn.

Along the Pacific Coast there is an excessive rainfall of from 100 to 150 in. per year, but it is distributed unequally throughout the seasons, and only by making use of storage lakes can extreme low-water conditions be avoided.

Until twenty years ago, hydraulic power often formed the basis of growing industries and determined frequently the location of the centers of population, but the larger centers of trade were developed by rail and water transportation, and in many cases were quite distant from such sources of power.

With the advent of electric power transmission, however, these cities offered a favorable market for the sale of light and power from distant sources, and private companies, and in some cases municipalities, have invested large sums in hydro-electric constructions and transmission systems to serve the adjoining districts.

While much has been accomplished, much remains to be done, and the discussion of present accomplishments which is herein given is intended to mark a mile-stone on the road.

Of the many great rivers of Canada, the Mackenzie, Nelson, and St. Lawrence are by far the greatest, having drainage areas of 660 000, 370 000, and 565 000 sq. miles, respectively; but, outside of these, there are three districts worthy of notice from a hydraulic standpoint, namely, the Atlantic Coast, the Pacific Coast, and that great array of rivers which, in addition to the Nelson, flow into the Hudson Sea.

Atlantic Coast.

On the Atlantic seaboard the configuration of the Provinces of Prince Edward Island and Nova Scotia does not provide large drainage areas, and, also, in the presence of extensive coal measures and coal mining, the incentive to hydro-electric development has been lacking; indeed, a recent installation of steam-electric equipment having a capacity of 500 kw. at the pit mouth, from which electric energy is transmitted at 11 000 volts to Amherst, N. S. (distant 6.5 miles), may be but the pioneer of much more extensive installations of a similar character in other parts of America.

Farther westward in New Brunswick, there exists the condition more usual in Canada, namely, rivers having large drainage areas,

PLATE X.
TRANS., AM. SOC. CIV. ENGRS.
VOL. LXV, NO. 1123.
SMITH ON
HYDRO-ELECTRIC POWER IN CANADA.

MAP OF
THE DOMINION OF CANADA

SCALE OF MILES
0 50 100 150 200 250 300

NOTE: Power Stations shown thus ⊛

and having their sources in swamp, forest, lake, and muskeg, in which snow and frost linger late in April, or even until May, and feed these rivers throughout the dry months of August and September; indeed, it is usual in Canada to find the extreme low-water period in March, before the frost has ceased to hold the moisture from flowing. The Miramichi, Restigouche, and St. John are great rivers, and extensive hydraulic constructions may be expected for grinding pulp alone, in addition to serving near-by towns and cities by transmitted electricity.

A most interesting and extensive development has been commenced at Grand Falls, N. B., by the Grand Falls Power Company, where the St. John River, with its sources in hundreds of lakes of Northern Maine, falls abruptly 115 ft. over a slate barrier.

For these hydro-electric works the type of construction which has been selected is that of vertical, 10 000-h.p., 3-phase, 25-cycle, 11 000-volt units, operating under a head of 130 ft., and the designs, under which contracts for the construction of the works are about to be let, provide for a hydraulic capacity of 80 000 h.p., with a present machinery installation of 40 000 h.p., which will generate current for ferro-manganese smelting, pulp grinding, and for transmission to the cities of Woodstock (70 miles), Fredericton (125 miles), and St. John (165 miles). An interesting feature of the design is that the generators are to be placed in the wheel-pit immediately above the turbines, with a provision for the proper ventilation of this subterranean chamber as well as careful water-proofing to ensure satisfactory electrical conditions. The tunnel tail-race will be placed entirely below the level of the tail-race water, and the draft-tubes from each unit, therefore, will be brought into the top of the tunnel from one side of the wheel-pit; this, it will be noted, is a variant on the methods adopted by the Toronto and Niagara Power Company in its works at Niagara Falls, Ont.

Pacific Coast.

On the Pacific Coast, the hydraulic conditions are very different from those in other parts of Canada, for the great mountain ranges are not good reservoirs, and, although the glaciers feed the rivers until late in the summer, yet at other seasons, owing to the light rainfalls east of the Coast Range, the rivers become very much reduced

in volume; thus the excessive floods of the Fraser River, for instance, make it unsuitable for hydraulic development for power purposes.

Near the Coast, however, the rainfall is excessive, and the existence of small lakes has facilitated the construction of several interesting developments, serving Vancouver and the adjoining territory. As instances, those of the Stave Lake Power Company and the Vancouver Power Company may be mentioned.

The works of the Stave Lake Power Company, now under construction, are 35 miles east of Vancouver, on Stave River, where a head of 90 ft. is to be utilized. The drainage area, estimated at 360 sq. miles, provides a large volume of water, owing to the excessive rainfall, which varies from 100 to 150 in. per year. The flow in the river is estimated to vary from 2 000 to 50 000 cu. ft. per sec., making the river regulation somewhat difficult.

The equipment, as at present designed, will consist of five units, each of 5 000 h.p., and it is considered that the electrical output from this installation has a good prospective market in Vancouver and Westminster to which the current is to be transmitted at an ultimate voltage of 60 000.

The Vancouver Power Company, Limited, has a most interesting development at Butzin Lake, 18 miles northward from Vancouver. This has been made possible by diverting the water of Coquitlam River at Lake Coquitlam into Butzin Lake by a 2½-mile tunnel having a section of 73 sq. ft.; thus the water required for operating the plant is conveyed from the Coquitlam drainage area, estimated at 100 sq. miles, the run-off varying from 300 to 5 000 cu. ft. per sec. under an excessive rainfall of 150 in. per year. At Butzin Lake the water is taken from the head-works by steel pipes to a power-house, 1 800 ft. distant, and located at sea level, 400 ft. lower.

The generating equipment is typical of the Pacific Coast, being originally composed of four units, each of 3 000 h.p.; two Pelton wheels for each unit drive a 3 000-h.p., 2 300-volt generator at 200 rev. per min., in addition to which there is also a recent installation of a 10 000-h.p. generator driven by four Pelton wheels, thus making an equipment having a total generating capacity of 22 000 h.p.

The electric current from this station is stepped up to 23 000 volts and transmitted to Vancouver, New Westminster, Burnaby, Lulu Island, and elsewhere, and is used for operating electric railways,

salmon canneries, car shops, saw-mills, and for general municipal and industrial purposes, thus serving the requirements of an urban population of about 90 000.

The transmission line consists of double towers each carrying two circuits of 3-phase, No. 2, B. & S., copper cable, and crosses navigable

FIG. 1.

water at one place where there is a span of 2 750 ft. suspended 150 ft. above the water.

The total single 3-phase circuit length of lines is 135 miles, and the transforming capacity at present in service is 14 650 kw. at the generating station and 16 400 kw. at the various sub-stations; all

these transformers are air-cooled, and are of different sizes up to a maximum of 2 500 kw. per phase.

In the interior of British Columbia the sparse population—almost entirely south of the main line of the Canadian Pacific Railway—is engaged either in agriculture or mining, and thus it happens that, aside from small developments at Revelstoke and Nelson for municipal purposes, the only interesting hydro-electric plant in this district is that of the West Kootenay Power Company, built to serve the mines and smelters at Rossland, Trail, and in the Boundary District. It is located at Bonnington Falls, on the Kootenay River, 9 miles west of Nelson, B. C. For the new power-station—which supersedes a smaller adjoining station previously built—the design is of the vertical type, owing to the considerable variations in tail-race level, caused by gorging below the falls in high-water periods.

The hydraulic head varies from 65 to 55 ft., being least at high water, and the minimum capacity of the river is estimated at about 6 000 cu. ft. per sec., being the run-off from a mountainous drainage area of nearly 10 000 sq. miles, which, owing to the glacial sources of the river, gives a low-water period in winter and a high-water period in summer. The station is a very interesting example of complete concrete construction, including even the intake channels and draft-tubes of the turbines, and is constructed for an equipment of four units, each of 6 000 h.p., nominal rating, with the turbines of 30% overload capacity. At present, two units are installed and a third has been ordered, in addition to which the old station has an equipment of 2 700 kw., and, when required, serves Rossland and Trail over a 31-mile, cedar-pole line at 20 000 volts on two No. 0, B. & S., 3-phase circuits.

At the larger station, the triplex, high-pressure, oil-thrust pumps, etc., are developed very much along the lines of the Niagara works, and are designed to supply oil to the thrust-bearings at 250 lb. per sq. in.; the governors were built by the I. P. Morris Company, of Philadelphia, adapted and modified from the well-known Escher-Wyss type.

The step-up transforming station supplies 60 000-volt current for the new cedar-pole transmission circuits to Phœnix Mines and Grand Forks Smelters, and to Greenwood Mines, distant 79, 69, and 83 miles, respectively; and its present equipment consists of six single-phase,

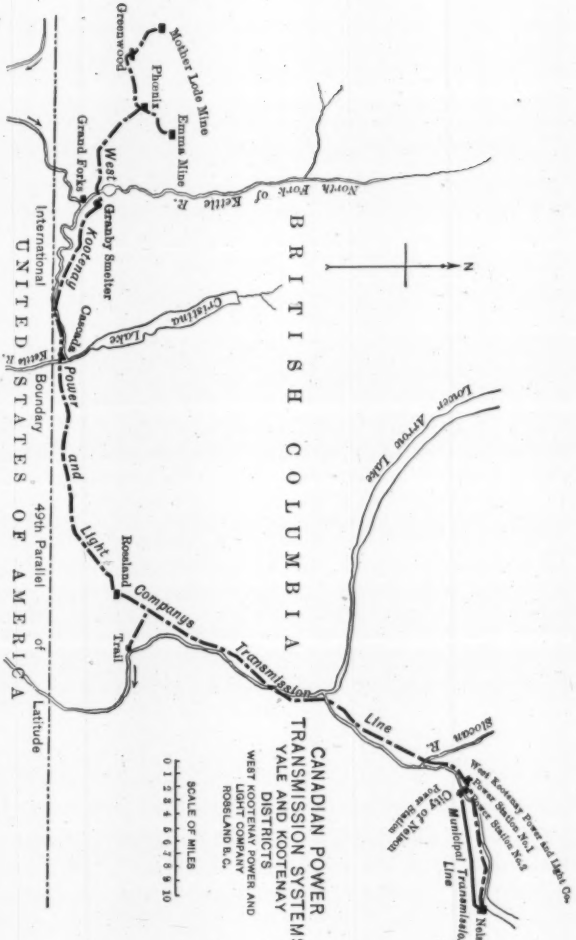

CANADIAN POWER
TRANSMISSION SYSTEMS
YALE AND KOOTENAY
DISTRICTS
WEST KOOTENAY POWER AND
LIGHT COMPANY
ROSSLAND B.C.
SCALE OF MILES
0 1 2 3 4 5 6 7 8 9 10
BRITISH COLUMBIA
UNITED STATES OF AMERICA
N
Greenwood
Mother Lode Mine
Phoenix
Emma Mine
Grand Forks
West Kootenay Power and Light Companys Transmission Line
Granby Smelter
Cascade
Kettle R.
Cristina Lake
North Fork of Kettle R.
Lower Arrow Lake
Slocan R.
Rossland
Trail
International Boundary
49th Parallel of Latitude
West Kootenay Power and Light Co.
Power Station No.1
Power Station No.2
City of Nelson Power Station
Municipal Transmission Line
Nelson

FIG. 2.

water-cooled transformers, each of 1 850 kw., nominal rating, each three-phase bank being in a fire-proof compartment; there is also a bank of three single-phase, 1 250-kw. transformers held in reserve for the Rossland and Trail Districts at 20 000 volts in the event of a shut-down of the older station.

The uses for which the electric current from this station is adopted, in this district, are quite varied, consisting of mine-hauling by electric locomotives, hoisting by induction and synchronous motors, lead smelting, handling of various machinery, pumping, air compression, and general industrial and municipal purposes.

The total transforming capacity in seven sub-stations is 31 000 kw., and the active connected load is 21 000 h.p.

Immediately east of this hydro-electric development are located the great Crow's Nest Pass coal areas, but the competition is not effective, in so far as service in the mining district is concerned.

It is also anticipated that some of the adjacent mountain railways will become electrified in the near future, as the operation of rack-railways on 4½% grades has not proven satisfactory or economical.

Perhaps the most interesting feature of these works is the satisfactory maintenance of a 60 000-volt transmission line over a heavy mountainous country on a substantial but ordinary cedar-pole line. This is in sharp contradistinction to the prevailing tendency in the more level districts of Eastern Canada, where it is considered good practice to use steel towers with long spans for high-voltage transmission, but doubtless the decision was influenced by the inaccessible nature of much of the country traversed, the plentiful supply of large cedar timber in British Columbia, and also the high freight rates on structural steel, to the interior, from the Pacific Coast.

The adjoining zinc and lead smelters at Nelson, B. C., will be supplied by these works or by the municipal hydro-electric works of the City of Nelson, on the other side of the river from the large power-station just described.

This municipal power-station has now a capacity of 5 000 h.p. and an equipment of 1 000 h.p., which is about to be increased to 2 500 h.p. by the installation of a second vertical unit. The general construction has been carried out on a basis of four units having a total capacity of 5 000 h. p.

The Great Plains.

Immediately east of the Rocky Mountains, in Canada, there are two great river basins, with many subdivisions, but of these the Mackenzie River, emptying into the Arctic Sea, lies in a practically uninhabited region; however, it is interesting to note that emigration and settlement will soon change all this, and, undoubtedly, in a few generations populous communities will occupy the upper water-sheds of this river, and will require all the tools of civilization, and among them the electric current from hydro-electric stations. Flour mills are already being established on the head-waters of the Peace and Athabasca Rivers.

The Nelson River Basin.

The Nelson River drains an extensive water-shed which is a country in itself. All the inhabited portions of the Provinces of Manitoba, Saskatchewan, and Alberta, except a small area tributary to the Missouri and Mississippi Rivers, are drained by the various tributaries of this river.

This water-shed is divided into three main drainage areas, the Saskatchewan, the Red and Assiniboine, and the Winnipeg Rivers. From a hydraulic point of view, each of these rivers exhibits distinct and dissimilar characteristics.

The Saskatchewan River, of 125 000 sq. miles drainage area, rising in the eastern slope of the Rocky Mountains by many sources, fed from glaciers, and descending to the plains through many mountain gorges, offers, near its sources, reasonable opportunities for hydraulic development, and two companies are already active in the preliminary stages of contracting to supply Calgary, Alta., with electric power from hydro-electric stations to be erected on the head-waters of the Bow River. Doubtless, as population in the foot-hills increases, similar undertakings will be carried out to meet the requirements of near-by towns and cities.

Eastward of the foot-hills, however, the conditions change, and the main branches become chiefly dependent on the precipitation on the great plains, which is rather light and either evaporates or passes off in spring or autumn floods. The character of the country also changes; the streams occupy deep, wide gorges cut down into the plains, and have light grades and shifting silt and sand bottoms where dam construction would be difficult and expensive.

Speaking generally, the main valley of the Saskatchewan, eastward from the foot-hills, is not adapted to hydro-electric constructions, and, as there is coal over a large proportion of this vast area or near it, it is not probable that such constructions will ever be numerous, extensive, or profitable, except at a point on this river immediately before it enters Lake Winnipeg.

At this place, 250 miles north of the City of Winnipeg, the river passes through a series of small lakes and then, at Grand Rapids, drops quickly to the level of Lake Winnipeg. The change in level is 112 ft. in a few miles, and there is thus afforded the opportunity for an enormous development of at least 300 000 h.p. whenever the demand arises for a power development at that point.

The Red and Assiniboine branches of the Nelson River, with a drainage area of 105 000 sq. miles, are still more unfavorably located for hydraulic purposes, as the drainage basin is entirely in a plains country, where there is light rainfall and very little lake storage, thus the minimum run-off is extremely small in proportion to the areas drained, as a consequence of which, with the exception of a few quite insignificant opportunities, they may be written off the list of hydro-electric districts, not only for the foregoing reasons, but because the flood levels are usually bank full, and sand and silt bottoms, over wide flats, present expensive conditions for dam constructions. It may be remarked in passing that this drainage area is devoid of coal, except for the lignites of the Souris Valley; and the price of fuel and power is excessive, except where the coal supply is local or transmitted electricity from the eastward is possible.

The Winnipeg branch of the Nelson River drains an area of approximately 55 000 sq. miles in the State of Minnesota and in the Provinces of Ontario and Manitoba, and, from its peculiar topographical features, affords one of the most favorable locations for the development of hydraulic power in Canada, excelled perhaps only by the Niagara and the St. Lawrence and superior in some respects to the Ottawa River.

This condition is somewhat, but not greatly, offset by the moderate rainfall, which is only from 20 to 25 in. per year over the main area, but, aside from this, there is a most extensive network of lakes (of which the location, throughout the northern portion, has been, in many cases, unmapped until recently). There are thousands of lakes

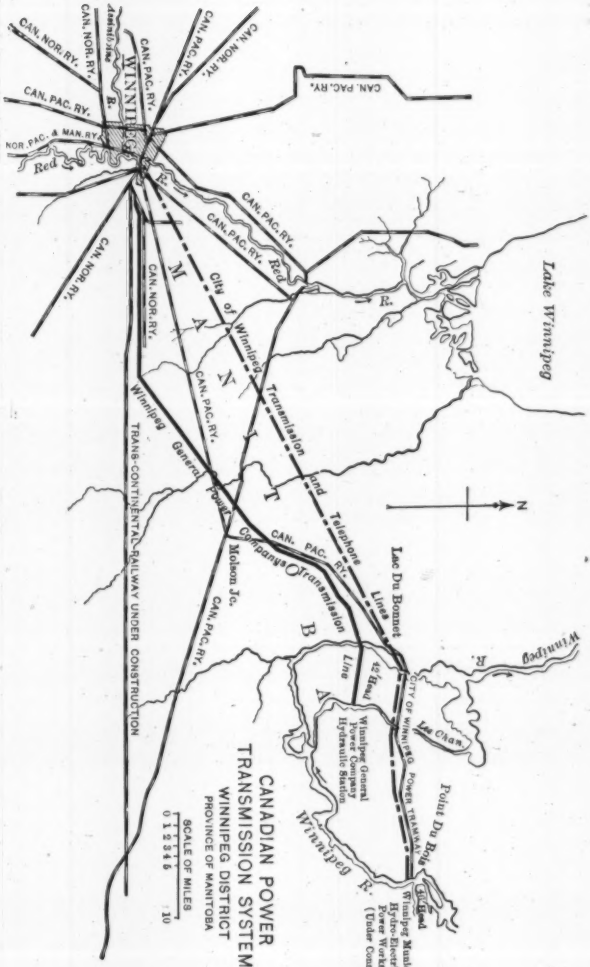

CANADIAN POWER
TRANSMISSION SYSTEMS
WINNIPEG DISTRICT
PROVINCE OF MANITOBA
SCALE OF MILES
0 1 2 3 4 5 10
WINNIPEG
Assiniboine R.
CAN. NOR. RY.
CAN. PAC. RY.
NOR. PAC. & MAN. RY.
Red R.
Lake Winnipeg
MANITOBA
City of Winnipeg Transmission and Telephone Lines
Winnipeg General Power Company Transmission Line
TRANS-CONTINENTAL RAILWAY UNDER CONSTRUCTION
Molson Jc.
Lac Du Bonnet
Winnipeg R.
Lee Chan.
Point Du Bois
42' Head
CITY OF WINNIPEG POWER TRAMWAY
Winnipeg General Power Company Hydraulic Station
Winnipeg Municipal Hydro-Electric Power Works (Under Construction)
N

FIG. 3.

of all sizes, culminating in the Lake of the Woods. There are 1 600 sq. miles of water surface on the Rainy River branch and Lac Seul on the English River branch, westward of which the two rivers unite near the eastern boundary of Manitoba and flow thence into Lake Winnipeg.

On the English River branch of the Winnipeg River no large hydraulic constructions have yet been considered seriously, except near Dryden, as the country is inhabited only at a few points along the line of the Canadian Pacific Railway, which crosses the various smaller branches between the height of land and the Town of Kenora, but the Transcontinental Railway, now being constructed, will pass much nearer the more extensive water-power locations of this river.

These locations, being only the more extensive ones of the English River branch, are rated by the Hydro-Electric Power Commission of Ontario at a total of 100 000 h.p., minimum dry-weather flow, 24-hour service, and it may be considered that the commercial capacity is at least 200 000 h.p. under ordinary conditions. It seems evident that the requirements of railway operation, flour milling, pulp and paper mill work, and possibly smelting will demand a careful study of the specific value of these powers in the near future, as soon as the Transcontinental Railway is in operation.

The Seine and Rainy River branch of the Winnipeg River also possesses good hydraulic opportunities, and the Koochiching Falls, at Fort Frances, is now being developed on a basis of 14 000 h.p., for which the dam construction is now completed and power construction is under way.

The Winnipeg River proper commences at Kenora, where the river leaves the Lake of the Woods, and from this point to the mouth of the river there is a chain of falls and chutes which, as sources of economical power, is difficult to surpass or equal.

The two conditions worthy of comment are that at each power location the rocky barrier creating the fall is admirable as a foundation for a dam, thus enabling constructions to be low in first cost, and at the same time creating considerable local storage to meet daily peak-load requirements. Also, the enormous lake storages, previously referred to, very much reduce the flood discharge and increase the minimum. The latter may be taken at 13 000 cu. ft. per sec. at Kenora and 20 000 cu. ft. per sec. below the junction of the English

River, the flood discharge being not more than four times these amounts.

Table 1 is a summary of the power sites on the Winnipeg River proper, between Kenora and the mouth at Fort Alexander.

TABLE 1.—Power Sites on the Winnipeg River.

Location.	Available head, in feet.	Minimum electric horse-power; 24-hour rating.	Average electric horse power; 24-hour rating.	Remarks.
Kenora	18	21 000	42 000	5 500 h.p. installed.
Island Falls	45	56 400	112 800	
Point du Bois	46*	83 600	209 000	20 000 h.p. being installed.
Slave Falls	30*	54 500	136 200	
Upper Seven Portages	30*	54 500	136 200	18 700 h.p. installed at Lee Channel.
Lower Seven Portages	22	40 000	100 000	
McArthurs Falls	12	22 300	55 750	
Grand Bonnet	30	56 000	140 000	
Little Bonnet	10	18 500	46 200	
White Mud Falls	15	28 000	70 000	
Silver Falls	21	39 100	97 800	
Pine Portage Falls	12½	23 300	58 300	
Totals		497 200	1 204 250	

* Including raising the water level by head-works.

Of the sites mentioned in Table 1, Island Falls, Point du Bois, Slave Falls, Upper Seven Portages, and Grand Bonnet afford excellent construction opportunities. All but the first two in Table 1 are within easy transmission distance of the City of Winnipeg, and all are within short distances of two transcontinental railway systems.

The situation at Kenora illustrates the great value of water-power as applied to flour milling. The town has little else to recommend it as an industrial center, not being at a competing railway point, nor at a bulk-breaking point; neither is there any surrounding population or any fertile areas; but, in spite of these apparently unfavorable conditions, three large flour mills are located there, and use about 4 000 h.p., with a capacity of 10 000 bbl. per day. Two of these plants operate privately-owned hydraulic equipments, and the third purchases 1 000 e.h.p., at a flat rate of $10 per e.h.p.-year, from the municipal power-station. The latter has been built recently at the eastern outlet of the Lake of the Woods with dimensions to receive machinery having a capacity of 4 000 e.h.p., one-half of which has already been installed.

At the main or western outlet of the Lake of the Woods, at Kenora, there has also been built for some years, a stone dam from which water could be drawn to develop approximately 18 000 e.h.p. at low-water, but no further constructions have been commenced.

At Point du Bois the City of Winnipeg (population 120 000) has commenced the construction of a large generating station involving a 75-mile transmission line to the city, and the designs contain some interesting features. A short (1 200-ft.) head-canal, concrete overflow walls, and a rock-filled dam in the main river will raise the water 13 ft., creating a total head of 46 ft. and forming a lake 5 sq. miles in area, thus producing uniform and quiet hydraulic conditions. The dimensions of all control works, dams, etc., provide for an ultimate equipment of 60 000 h.p. The remainder of the work is to be a solid reinforced concrete structure involving an enclosed forebay room, power-house, and transformer-station equipped primarily with five 5 300-e.h.p. units, which will consist of horizontal, double-runner turbines (draft-chests only) in reinforced concrete wheel-pits and directly connected through a breast wall to 3-phase, 60-cycle, 6 600-volt, generators making 164 rev. per min.

The transformation will be to 77 000 volts, by water-cooled, oil-insulated transformers having a capacity of 3 750 kw. per phase. Similar transformation at Winnipeg will be from 72 000 volts down to 11 000 volts in a main terminal station, from which 11 000-volt feeders will distribute to three or more sub-stations for local distribution at suitable voltages; the transformers at the terminal station, however, are to be oil-circulating, and oil-cooled. The system of remote control will be fully applied at both stations.

The transmission lines, of No. 000 copper, or aluminum equivalent, will be carried on double-circuit steel towers spaced 600 ft. apart, each alternate tower being of elastic type with two legs only. All tower foundations are to be on concrete.

The city utilities are already numerous, and include Artesian water supply, pumped electrically; and the desire to be able to offer cheap power to industries, present and prospective, which would otherwise labor under a great disadvantage in a city which cannot offer bituminous coal of a good quality for less than $6.50 per ton, while anthracite coal is $10.50 per ton retail, appears to be ample justification for this ambitious project for a city of 120 000 inhabitants.

The Winnipeg Street Railway Company and its allied companies have in operation a hydro-electric station on the Lee Channel of the Winnipeg River, and a transmission line 67 miles long supplying power to a terminal station in the city.

This generating station was selected apparently without proper investigation and without an eye to the future, and the company is already expending large additional sums on the tail-race channel in order to insure sufficient hydraulic head and flow of water at low water to operate the machinery now installed, which makes a completed scheme incapable of further expansion and consisting of nine units having a combined rated capacity of 14 000 kw.

The scheme of development consists of a rock-filled dam across the Winnipeg River, which raises the normal water level 4 or 5 ft. and forces a flow of water through a 14-mile high-water channel, from which already approximately 500 000 cu. yd. of rock have been excavated.

The power-house is about 5 miles from the intake of this channel, and was designed for an operating head of 42 ft.; this, however, has not yet been realized, and it is understood that, when delivering 15 000 e.h.p. at Winnipeg, the generating station is only operating under a head of 36 ft., and is at its present limit of capacity, hence the further excavations now in progress.

The generating station has an open forebay, steel-flume wheel-pits, and units of five 2 000 kw., and four 1 000 kw., each operated by two twin horizontal turbines; the generators are for 3-phase, 60-cycle, 2 300-volt current.

The transformation is to 50 000 volts, and the transformers are oil-insulated, water-cooled, of 1 800- and 830-kw. capacity per phase, each being in a separate compartment. The total transformer capacity at the generating station is 19 500 kw. The transmission system is on double-circuit, steel towers, spaced 500 ft. apart, with protruding pieces of angle iron on each upper corner of each tower which act as lightning arresters. It is understood that this protection is very satisfactory, there having been only one shut-down during the first year's service (which commenced in 1906). The capacity of the terminal station in Winnipeg is equal to that of the transforming station at the generating works.

The utilities served consist of the city and suburban street rail-

ways, the city house-lighting, and current for the present municipally-operated street lighting. In addition to this, there is a small distribution of power for miscellaneous purposes, and two large blocks of power for flour milling. The capacity of the generating station is already fully taxed during the low-water winter season, and no effort is made to encourage large users of power by offering attractive rates; this power-station cannot be looked upon as a serious competitor to the municipal power-station now being built.

The three tributaries of the Nelson River, which have just been discussed hydraulically, converge in Lake Winnipeg and there form an enormous reservoir of 9 300 sq. miles at an elevation of 710 ft. above the Hudson Sea.

When it is considered that the drainage area feeding the outlet of Lake Winnipeg into the Nelson River proper is nearly 350 000 sq. miles, and that the water supply comes from three different and dissimilar climatic and topographical river basins, one can appreciate what a large minimum flow there is; and, as the route of the river to the Hudson Sea is rock-ribbed, in cascades, pools, and waterfalls, all that need be said here is that, should electro-chemical or other demands require very extensive water-powers, where relation to habitation is immaterial, these, to the extent of at least 4 500 000 h.p., can be readily secured on the Nelson River between Lake Winnipeg and the Hudson Sea.

Other Hudson Sea Rivers.

Of the many extensive rivers flowing into the Hudson Sea and James Bay, there is very little accurate information, and hydraulically these rivers are almost unknown except in a general way.

It can be appreciated, however, that, from the physical conformation, many enormous water-powers exist, because the table-land sources of the rivers abound in lakes at an elevation of from 600 to 1 500 ft. above the sea, and this, taken with a good rainfall and the rocky nature of the country, warrants this certain assumption. It is generally known, however, that near the sea coast most of these rivers are alluvial, and that the water-powers of value lie considerably inland, in general, at least 200 miles from the mouths.

The first development, north of the height of land which lies between the St. Lawrence River and the Hudson Sea, will probably be at Iroquois Falls, on the Abitibi River. The construction of the

Ontario Government Railway, parallel to the river, and that of the Dominion Government Transcontinental Railway east and west immediately north of Lake Abitibi, suggest the early development of Iroquois Falls for purposes of pulp grinding, or power for mines, or for general distribution to the junction city which will immediately grow up in that vicinity.

The St. Lawrence Basin.

The St. Lawrence River drainage basin is the chief seat of population in Canada, and naturally the hydro-electric developments of this region are most extensive.

The total drainage area is 565 000 sq. miles, but of this about 90 000 sq. miles comprise the water surface of the Great Lakes, which form enormous compensating reservoirs, their regulation and control, international in character, presenting a very delicate physical and political question, additionally complicated by the rights and needs of navigation; therefore it is advisable to discuss the water-powers of the main river and of its many important branches separately.

Sault Ste. Marie.—The first water-power location on the main river is at Sault Ste. Marie, where Lake Superior, with 31 000 sq. miles of water surface, empties into Lake Huron, falling approximately 18 ft. and discharging about 60 000 cu. ft. per sec. at low water, thereby affording opportunity for about 90 000 e.h.p., after making allowances for various hydraulic losses. The present developments have been somewhat international in character, and have been carried out chiefly by the Lake Superior Power Company, which, with allied companies, disposes of 6 500 h.p. of hydraulic and electric power on the Canadian side of the river (out of a designed total of 20 000 h.p.) to rail mills, pulp mills, and for industrial and municipal purposes in the Canadian city.

On the United States side of the river the same company constructed a canal and power-house, under unfavorable physical conditions, designed to obtain 45 000 h.p.; but machinery for only one-half of this, or less, has ever been installed, and the chief customer, outside of municipal and small industrial users, is a carbide company having works in the vicinity; a third company, on the United States side, has a small hydro-electric development, but it is probable that no further concessions will be granted there, or, if granted,

will be under the control of the International Deep Waterways Commission, so that the needs of navigation, both as regards the maintenance of the level of Lake Superior and the water required in increased quantities for operation of the three locks at this point, may be fully conserved.

It is worthy of more than passing comment that, although these canals and locks serve an enormous commerce, their existence has been of little local value, whereas the life of the two adjacent cities of Sault Ste. Marie depends chiefly on the creation and operation of hydro-electric power, and the two Governments interested will be well advised to use every effort to facilitate rather than hinder the power use of all the water passing from Lake Superior, which is not required for locking purposes. It is evident that control works at the head of the rapids can be constructed so as to compensate for any quantity of water which may be diverted and used for power purposes; and, geographically, as an assembly point for raw materials by water and rail, Sault Ste. Marie has many advantages.

Niagara Falls.—Continuing seaward from Sault Ste. Marie, the next location which affords hydraulic opportunity is spoken of familiarly as Niagara Falls.

Lake Erie flows into Lake Ontario by a river route which, from Chippewa to Queenston, is a series of rapids and the cataract, but the cataract of Niagara Falls is barely one-half of the total difference in elevation, which is 327 ft., and that the first great hydro-electric developments have been made there with effective heads of from 136 to 175 ft. can only be looked upon, in the future, as an economic blunder, because there are potential developments which can be made at several locations in the United States and Canada using a net head as great as 300 ft., which, for as large an installation as 200 000 h.p. or greater, would not be as expensive as those now constructed, and the deduction is here offered that when industry makes the demand neither international understandings nor the makeshifts of opportunist politicians will prevent capital from carrying out the construction of these undertakings for the benefit of mankind.

This question of utilizing the hydraulic power of the Niagara escarpment is, in reality, a world problem, not a local one, because there is there latent 5 000 000 continuous electrical horse-power, considering only the low-water flow of 180 000 cu. ft. per sec. at an

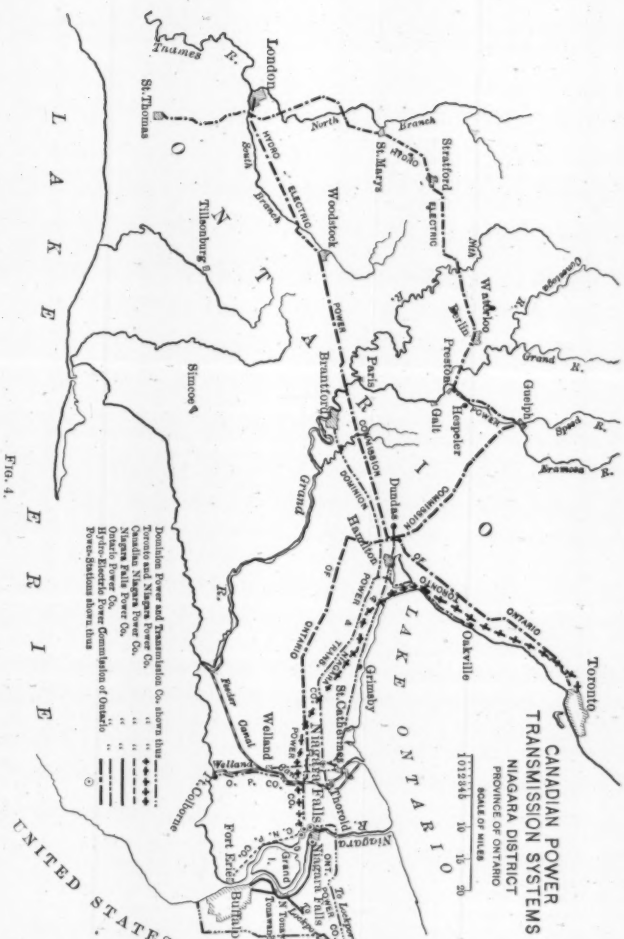

CANADIAN POWER
TRANSMISSION SYSTEMS
NIAGARA DISTRICT
PROVINCE OF ONTARIO
SCALE OF MILES
1 0 1 2 3 4 5 10 15 20
LAKE ERIE
LAKE ONTARIO
ONTARIO
UNITED STATES OF AMERICA
London
St. Thomas
Tillsonburg
Simcoe
Woodstock
St. Marys
Stratford
Waterloo
Berlin
Preston
Galt
Hespeler
Guelph
Paris
Brantford
Dundas
Hamilton
Oakville
Toronto
Grimsby
St. Catherines
Welland
Niagara Falls
Thorold
Fort Erie
Buffalo
Tonawanda
Niagara Falls
Dominion Power and Transmission Co. shown thus
Toronto and Niagara Power Co.
Canadian Niagara Power Co.
Niagara Falls Power Co.
Ontario Power Co.
Hydro-Electric Power Commission of Ontario
Power-Stations shown thus

FIG. 4.

effective head of 300 ft., and, estimating even so little as 2½ lb. of coal per horse-power-hour, this would be equivalent to a saving of 55 000 000 tons of coal per year, should this great force of Nature ever become so fully utilized; and why should it not be used in increasing amount as required? There will still be available for scenic purposes all the surplus water, which will still pass over the falls, even after the minimum has all been used for power purposes. Consider the question otherwise; this location is unique, as it is on one of the greatest highways of commerce in the world, both water-borne and by rail, and it is in the midst of a fertile and populous region, which, for 150 miles in all directions, is becoming more thickly populated year by year. The steady growth in the electric load carried by the present stations leads to the inevitable conclusion that we are only in the infancy of the utilization of this great gift of the Creator.

The level of Lake Erie can be readily controlled by compensating works at the head of the rapids, and might be designed so as to constrict the natural channel at the same rate as the artificial channels were created, either near the river or by other routes from Lake Erie to Lake Ontario, and this arrangement appears to meet and answer the only serious criticism which has tended to array the shipping interests in opposition to further utilization of the water for power purposes; and, indeed, diminution of natural flow presents certain positive advantages, as it will check that rapid erosion of the Horseshoe Falls which is now taking place.

Recurring to the actual constructions which have been made since the first station of the Niagara Falls Hydraulic Power and Manufacturing Company was completed, on the United States side of the river, in 1881, it may be briefly stated that there are six large generating stations, two on the United States side and four on the Canadian side of the river, engaged in producing electrical energy for commercial purposes, and, in addition to these, there are numerous small stations, one at Niagara Falls, Ont., and several along the route of the Welland Canal, which produce a total of probably 15 000 h.p. for miscellaneous electrical and hydraulic purposes.

It is unnecessary to describe in detail the diverse constructions, distributions, etc., which have been described so often and so minutely by various engineers, but certain broad facts will be given by which the scope of present accomplishment may be gauged.

On the United States side, the Niagara Falls Hydraulic Power and Manufacturing Company has the distinction of being first in the field, and, from diminutive beginnings in 1872, it has expanded its works until at the present time it has 77 500 e.h.p. installed, and building construction is under way for additional equipment having a capacity of 70 000 e.h.p. In addition to this, water is supplied to various industries developing 8 000 h.p. of direct hydraulic power for pulp, paper, flour, and other mills.

The power from these works is used for street railway and various industrial and electro-chemical purposes, but chiefly for the manufacture of aluminum, by The Aluminum Company of America, which company has here its largest works. It is understood that this company contracts for nearly 60 000 h.p.

The head canal for this development is 110 ft. wide and 14 ft. deep at normal water level. It is constructed through the City of Niagara Falls, N. Y., for a distance of 4 400 ft., and terminates at a forebay immediately overlooking the river. The water is used under a total head of 210 ft. and a net head of about 200 ft., and is led down the side of the cliff in steel penstocks to the power-stations at the lower river level, where is installed a heterogeneous collection of direct-current and alternating-current generating units composed of various sizes from 150 to 1 000 kw. in the older station and from 3 500 to 6 500 kw. in the newer station, which facts well illustrate the growth of the art and the magnitude of the business of this station, which is entirely local, involving no transmission lines of any importance.

The Niagara Falls Power Company has two generating stations operating under an average effective head of 140 ft., one on each side of a short head-canal, and the common tail-race is a brick-lined tunnel passing to the lower river, 7 000 ft. distant.

These stations, heralded at the time as pioneers, have long since been eclipsed by the stations on the Canadian side of the river, and also suffer from certain engineering defects which, no doubt, were unforeseen at the time of construction and have since been cumulative.

The ice difficulties, after years of strenuous effort, have been largely overcome, by improvements in the intake works and by the addition of an ice shoot to the tail-race tunnel, but the first station, having a capacity of 50 000 h.p., has unprotected ice racks and is equipped with free, outward-discharge, double-runner, vertical tur-

bines. The ten generators are each 5 000-h.p., 2-phase, 2 200-volt, with revolving armatures. The second station, having a capacity of 60 000 h.p., has additional submerged-arch entrances and an enclosed forebay for further protection from ice. The turbines are of the single-runner, vertical type, with draft-tubes; the eleven generators are each 5 500-h.p., 3-phase, 2 200-volt, and are partly of revolving-field and partly of revolving-armature types. The tail-race tunnels are led from the power-stations, by very sharp curves, to a junction, and this, together with the back-piling in Power-house No. 1 due to the use of free-discharge wheels, makes it impracticable to run both stations to full capacity. It is understood that 83 000 h.p. is about all that is generated, although the market in Buffalo is being largely abandoned to the Canadian Niagara Power Company, its station being several miles nearer and belonging virtually to the same financial interests.

The power from these stations is now distributed in large blocks, chiefly to local electro-chemical consumers, and the policy of the company in creating a manufacturing district, with railway switching facilities, building sites, etc., and giving long-term power contracts, with guaranties of continuity, under penalty, has created a large and stable industrial center consuming more than 60 000 h.p. continuously from those two stations. The power transmitted to Buffalo, Tonawanda, etc., is 22 000-volt, 3-phase, 25-cycle. The pole lines are of ordinary wooden type and carry four 3-phase circuits, two on each line, some circuits being of copper and some of aluminum. Until the load was transferred to the Canadian station, all the varied electrical utilities of Buffalo, to the extent of about 40 000 h.p., were operated from these United States stations, as are still the various interurban and scenic electric railways converging at Niagara Falls.

On the Canadian side of the Niagara River four large generating stations have been constructed recently; three of these are at Niagara Falls, and the fourth is at a point 12 miles westward.

The Dominion Power and Transmission Company (formerly the Hamilton Cataract Power Company) first commenced to deliver power to Hamilton, Ont., in 1897, from a small station 12 miles westward of Niagara Falls at the foot of the Niagara escarpment, and drawing its water supply from the Welland Canal and thus from Lake Erie.

This hydro-electric development utilizes a hydraulic head of 267 ft.,

and is a very economical one, as excessive expenditures for head-works were not required. This enterprise has a progressive management, and a growing market for power has expanded it until, at the present time, the machinery equipment, including transformers, has a nominal rating of 38 400 h.p.

It is an interesting feature of this generating station that the turbines are entirely of European manufacture and design, being supplied by Italian and German makers, as the earlier experiences with manufacturers in the United States were not satisfactory.

The electric current is generated at 2 400 volts, 66⅔ cycles, and is transmitted to the surrounding towns and cities at various voltages. The chief market is in Hamilton, 32 miles westward, to which three pole-line, 3-phase circuits, two of copper and one of aluminum, carry power at 40 000 volts. In addition to this, however, the company serves a network of street and interurban railways which it owns or controls, and supplies power and light in Brantford, Welland, St. Catharines, Dundas, etc., with a total of 132 miles of transmission line.

This company, owing to its favorable investment and strongly entrenched position, is able to distribute with profit at low rates for power, and has thus built up a large business very rapidly. It is understood that the peak load in Hamilton alone, which is a city of only 60 000, is considerably more than 20 000 e.h.p., which includes current for operating street and interurban railways owned by this company, and also cotton mills, knitting mills, flour mills, rolling mills, and machine shops; in fact, every phase of a busy manufacturing city is provided for.

This company also owns small hydro-electric generating stations at St. Catharines and Brantford, and may be said to control, almost exclusively, the business of supplying electricity in the Niagara District west of Niagara Falls and Thorold.

The generating station of the Canadian Niagara Power Company was constructed between 1901 and 1905. This company is practically identical with that of the Niagara Falls Power Company, and, although it distributes some 2 000 to 3 000 h.p. locally to various industries in Niagara Falls, Ont., it has not entered into the long-distance transmission field, no doubt considering the local sale to be ultimately more profitable. The generating station is of the wheel-pit type, with a tail-race tunnel 2 200 ft. long, and operates under an

effective head of only 140 ft. The water is taken from the rapids immediately above the cataract, and although protecting works, submerged-arch entrances, enclosed forebay room, and sluiceways have been constructed to meet the difficult ice conditions, they are only moderately satisfactory, and a continuous winter service is maintained only by strenuous efforts and with considerable difficulty.

The generating works are complete for eleven units, each having a nominal rating of 10 000 e.h.p., and five of these machinery units are now installed.

The turbines, of Swiss design, are of the vertical type, with draft-tubes and double runners, and the generators are 12 000-volt, 3-phase, 25-cycle.

This station is inter-connected with those of the Niagara Falls Power Company by a large number of 11 000-volt underground cables, thus warranting great certainty of service; but the chief market for power is in Buffalo.

The company has a transformer station, adjoining the generating station, with an equipment of fifteen single-phase, 1 250-kw. transformers, all mounted in one room; and although these are arranged so as to be operated, if desired, at various voltages up to 60 000, the pressure used at present is 22 000.

The transmission system consists of steel poles, carrying two 3-phase circuits of aluminum cable mounted on cellulose insulators, and a third circuit is now being built. The capacity is 12 500 h.p. per circuit at 22 000 volts. At Fort Erie this transmission system has two spans of 1 600 and 2 200 ft., respectively, suspended 130 ft. above the water of the Niagara River, and it is understood that considerable difficulty was experienced in cable breakage until elastic counter-weighted suspensions were made at the end towers with suspended insulation at the center tower.

Under the franchise conditions existing between this company and the Provincial Government, and having in view the water limitations now being reported on and recommended by the International Deep Waterways Commission, it would appear to be inevitable that this company will soon be forced to enter actively into exploitation of the Canadian market, in order to enable it to increase its output sales beyond the present, which not only amounts to about 40 000 h.p. from its own station, but to this must be added some 5 000 h.p. which is

being obtained from the Electrical Development Company and exported.

It is interesting to note that the combined output of the three stations of these allied companies, *i. e.*, The Niagara Falls Power Company and the Canadian Niagara Power Company, is more than 500 000 000 kw-hr. per year, and that the load factor is more than 70%, owing to the large number of electro-chemical customers.

Between 1902 and 1906, the Ontario Power Company constructed, immediately below the cataract, a station which is equipped at present with six horizontal units, each of 10 000 or 12 000 e.h.p., generating 3-phase, 25-cycle, 12 000-volt current, at 187½ rev. per min., under a net head of 178 ft. The turbines are of German design and manufacture. The water is brought to the station by an 18-ft. steel pipe, 6 500 ft. long, which is embedded in concrete just below the hydraulic grade line, and buried in Queen Victoria Park a few feet beneath the surface.

This company selected a favorable location for its entrance works and forebay, which are located at the head of the rapids and are completed for a capacity of 180 000 h.p., and, as far as ice troubles are concerned, it may confidently expect to give a continuous service.

In several other features these works are interesting, particularly in respect to the methods of control. To handle successfully an 18-ft. column of water, 6 500 ft. long, moving at 15 ft. per sec., under ordinary commercial electrical conditions, was no mean task, but it is understood that, by means of a spiral overflow structure, this has been accomplished. The electrical control is centered in the transforming and distributing station, which is 600 ft. from the generating station and on the high bluff overlooking the cataract. At this point all the current from the generators to the high-tension lines is controlled. The transforming station is equipped with eighteen single-phase, oil-insulated, water-cooled transformers, each of 3 000 kva., each bank being in a separate compartment. This company has created several allied companies which distribute 62 500-volt and 12 000-volt current in the United States and Canada. Its extreme eastern delivery is at Syracuse, N. Y., 161 miles distant, and it distributes current westward to St. Catharines, Welland, and Port Colborne; but in view of the contract it has now entered into with the Hydro-Electric Power Commission of Ontario for the supply of from 25 000 to 100 000 h.p. at from $9.00 to $9.40 per h.p. per year for 12 000-volt continuous power,

it appears evident that its sphere of influence in Southwestern Ontario is in the near future to become very considerable, and the output of its station increased to at least 50 000 h.p.

At present its maximum load is 25 000 h.p. in the United States and 5 000 h.p. in Canada, sales being at the rate of 110 000 000 kw-hr. per year.

This company's transmission lines are very extensive and varied in character, and comprise several hundred miles of double-circuit and single-circuit, aluminum lines, chiefly mounted on steel towers carrying one circuit each. Some of the branch lines and 12 000-volt lines, however, are on timber, A-frame towers or cedar poles. The various constructions have been carried out substantially and scientifically, but are subject to the criticism of transmitting at only 60 000 volts for distances up to 160 miles, whereas, at the present time, 80 000-volt transmissions are in use and 100 000-volt projects are under construction; however, it is true that the suspended insulator has not yet been fully developed to unquestioned satisfaction, and, at 100 000 volts, air emanations from conductors, line troubles, etc., are certain to be considerable.

In 1903 the Toronto and Niagara Power Company was formed by parties interested in the Toronto Street Railway Company and the Toronto Electric Light Company, and, after negotiations to buy electric current from other Niagara companies had failed, a lease was secured from the Provincial Government to build a generating station in Queen Victoria Park at Niagara Falls at a site lying between those of the Canadian Niagara Power Company and the Ontario Power Company, the works of both these companies being then under construction. The location chosen was notable in two particulars, the intake being almost wholly in the surging rapids above the cataract, while the tail-race tunnel was located so as to emerge beneath the cataract itself. The construction of these works proceeded from 1903 to 1907.

Preliminary to construction, it was most difficult to construct the coffer-dams and keep them water-tight, and, together with the extensive overflow dam, etc., afterward constructed, made a heavy investment necessary. The wheel-pit is constructed for eleven units, each of 8 000 kw., and four of these units are now installed. The operating head is about 140 ft. and the station is equipped with

Francis double-runner turbines, constructed by United States manufacturers on Swiss models. In various items, however, these turbines differ from previous constructions. The draft-tubes are carried outward to twin, parallel, tail-race tunnels, and emerge from underneath so as to maintain the water seal. The penstocks are alternately right and left, in groups of two, thus economizing in wheel-pit length. The supporting structures for the vertical main shafts of the units are concrete arches instead of steel girders. Altogether, these works give evidence of advance in the art of wheel-pit construction. The power-house is a most ornate structure; indeed, this company has carried on all its operations with a lavish hand, and a resulting large investment. Its generating company is known as the Electrical Development Company, and the 3-phase, 25-cycle, 12 000-volt current generated is delivered outside of Queen Victoria Park to a transformer station whence the Toronto and Niagara Power Company transmits it at 60 000 volts to Toronto, 80 miles distant (population 300 000), over a double-circuit steel-tower line. This line is carried across Hamilton Beach and Canal and the Welland Canal by pairs of very high and heavy towers; otherwise the transmission line passes through a level cultivated country without serious transmission difficulties, although including various spans across small valleys of more than 1 000 ft. each.

The transforming stations are built apparently with a great effort to attain safety, each single-phase transformer being in a separate compartment, and there is very extensive concrete cell construction for bus-bars, lightning-arrester compartments, etc. The Toronto terminal station also has an auxiliary water-tower for supplying water to transformers in addition to the water service from the city mains. The current delivered at this station at 55 000 volts is stepped down to 11 000 volts and is then carried by underground feeders to various sub-stations in the city where it is transformed into 600-volt, D. C., and various voltages, A. C.; also, the frequency is changed from 25 to 60 cycles. Altogether, from 20 000 to 22 000 e.h.p. are used by the two subsidiary companies, in addition to sales at Niagara Falls to the Canadian Niagara Power Company for export, as before mentioned, and there are also subsidiary companies formed to distribute eastward in the United States.

This brief description of the capacities of the various Niagara

generating stations—amounting to 450 000 e.h.p. in Canada, when the stations described are fully equipped—and the fact that several other companies own charters and water rights, and are anxious to enter the field if given opportunity and encouragement, at first, would suggest that electric energy can now be obtained in this favored district at very low rates, and, comparing the rates offered with the cost to any given consumer to produce power or light by steam-driven prime movers, certainly the rates do improve these conditions somewhat, as is shown by the steady growth of electrically-operated industries in Hamilton, Ont. Moreover, from $20 000 000 to $25 000 000 have already been invested by these four companies in generating stations and transmission systems which must find a return by the sale of the electrical output, but, for the last ten years, the people of Southwestern Ontario have steadily asserted that, to create a great industrial area in that part of the province, still cheaper power ought to be and could be obtained.

Boards of trade began to agitate in 1900, and in 1902 a committee, from various cities and towns, was formed to take concrete action, which, by pressure on the Provincial Government, induced the passage of an Act in 1903, enabling two or more municipalities to appoint a Commission and, after investigation, to carry out, if desired, the construction of the necessary works at Niagara or elsewhere for generating, transmitting, and distributing electric energy.

This municipal commission was appointed, made a very exhaustive investigation, and reported in 1906 to the seven municipalities then represented in the movement, but, owing to the limitations evident in the working of the Act, and to a larger movement having commenced, there has been no specific outcome from this report.

In 1905 the Provincial Government, becoming more fully alive to the importance of this question, appointed a Government Commission, which was instructed to study and report on the hydraulic possibilities in the Province, the industrial demand then existing for electric energy, and the prime cost of generating and delivering the same to the various industrial centers and customers. The reports arising from the investigations were presented, in due course, to the Legislature during the session of 1905-06, and, although the facts elicited furnished a comprehensive idea of the industrial condition then existing, and the cost of satisfying the same, and also, by comparison,

set forth the rates offered by distributing companies, it has only been after a subsequent protracted struggle that definite results have been obtained and the evident will of the people carried out.

The vested interests immediately attacked the reports and the *bona fides* of those who drew them up, but the Provincial Government, recognizing the demand made upon it, passed a very comprehensive Act, appointing a permanent Government Commission. In this commission was vested the rights of the Crown, and it was authorized to act in response to the demands of the municipalities and in their behalf.

This Commission, after 2½ years' existence, has justified itself; and a group of sixteen municipalities has made a demand upon the Commission to secure electric power, to construct transmission lines from Niagara, to transform the electric energy thus transmitted at the various sub-stations, to administer the construction, and to operate the works for a period of 30 years, over which period a sinking fund will be created out of the rates to repay the investments. The amount of power to be distributed immediately is approximately 30 000 e.h.p., but it is confidently expected that this will increase very rapidly to probably 100 000 e.h.p. in 5 years.

The various municipalities have voted the money necessary for local distribution, and the Government has recently contracted for the construction of 300 miles of transmission lines, and this work is to be followed immediately by the construction of the necessary sub-stations and local distributions.

This popular movement has yet to be justified by capable and honest administration, deficiency in which is the weakness of many municipal movements in America, and it is to be hoped that the ægis of the Government Commission will render the inherent merits of the project effective.

The large area of country to be covered and the necessary anticipation of a still wider area as soon as the first installation has justified itself, have forced on the Government engineering staff a consideration of higher voltage, and, according to present plans, the ultimate voltage of transmission now being provided for is 100 000. This advanced practice, however, has been adopted by reason of the necessary provision for further extensions of the Hydro-Electric Commission's system to points still more distant from the generating station

than London and St. Thomas (140 miles). The world will, doubtless, watch closely the results obtained in carrying out this huge municipal-provincial distribution of electric energy.

St. Lawrence River.—The St. Lawrence River, between Lake Ontario and Montreal, may be considered as having its navigation interests paramount, and, although attempts have recently been made to obtain permission from the Canadian Government to devote the whole river to power development, it is not likely that such permission will ever be granted.

In the meantime, along the ship-canal system, many small installations have been permitted, but they are chiefly local in character, and are devoted to operating mills and factories. They amount roughly to 10 000 h.p., and, in addition to this, hydro-electric stations having a capacity of about 5 000 h.p. supply the various near-by towns with light and power, and light the ship canals during the navigation season.

The whole river, if available, would be a great source of power, as it is capable of developing, at low water, 1 500 000 h.p. of electric energy at the various rapids.

Montreal.—In the vicinity of Montreal (population 400 000), several interesting constructions have been made at various times by various companies, but finally all these interests have been merged into the Montreal Light, Heat and Power Company, which controls the situation at present, although a rival company has recently been organized to develop power on the abandoned Beauharnois Canal of the St. Lawrence system and transmit to Montreal and distribute in opposition to the Montreal Light, Heat and Power Company. The latest construction by the latter company is at Soulanges, on the Soulanges Canal of the St. Lawrence River, 27½ miles west of Montreal, where 40 000 cu. ft. per sec. are available under a head of 54 ft. The equipment now being installed consists of three units, each of 3 750 kw., which will generate 3-phase, 60-cycle current at 4 000 volts. The transmission will be on one circuit at 44 000 to 40 000 volts, and the transformation will be made by banks of single-phase, water-cooled transformers having a rated capacity of 2 500 kw. per phase.

At present, this company owns two water-power stations and four steam stations, the latter having a capacity of 9 000 h.p. The com-

pany also purchases 16 000 h.p. delivered in Montreal by the Shawenegan Water and Power Company.

The station at Lachine, on the St. Lawrence River, is an example of daring but mistaken construction, carried out 12 years ago, when the transmission problem had not been as fully worked out as at

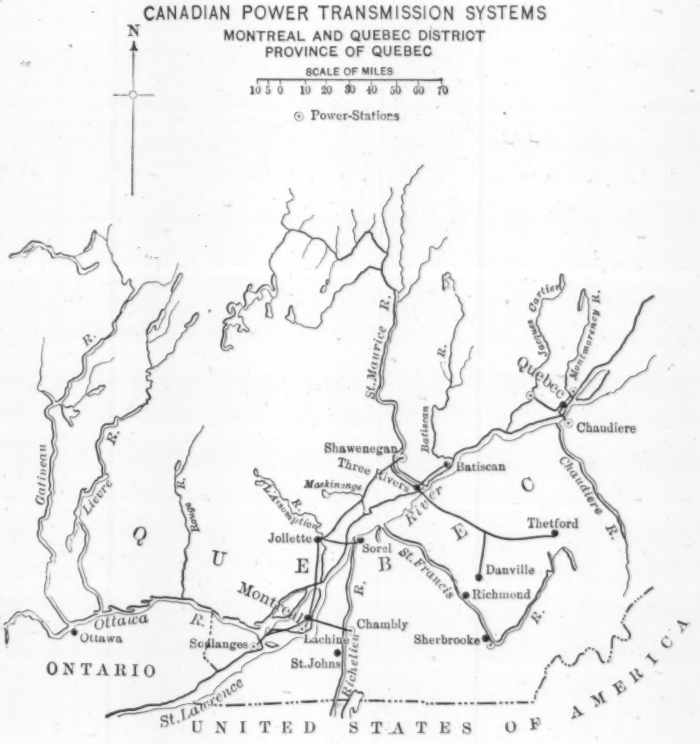


FIG. 5.

present. The works are really constructed as a part of the river, and therefore, in operation, they are subject to troubles from frazil in the head- and tail-races; but, after excessive expenditures, made for the purpose of creating quiet conditions above the power-house and protecting the tail-race from back-water, the station can be reasonably depended on to give continuous winter service.

The dependable hydraulic head is only 14 ft., and the eight generators, each having a rated capacity of 750 kw., are operated by groups of six turbines each, driving jack-shafts.

The current generated is 3-phase, 60-cycle, at 4 400 volts, and is transmitted over eleven circuits at this voltage direct to Montreal (6½ miles), where it is stepped down to 2 400 volts for distribution. The station was designed for a capacity of 16 000 h.p., but it has never been equipped with machinery beyond 8 000 h.p.

At Chambly, on the Richelieu River, a branch of the St. Lawrence, 18 miles southeast of Montreal, the company owns a generating station which has also had a history interesting to engineers. The concrete dam, constructed to create a head of 34 ft., failed soon after completion, and the first year's operation was much disturbed by frazil, but a new dam has since replaced the faulty structure, and the station now operates normally.

The equipment consists of eight units, each of 1 800 kw., driven by four turbines per unit, the total generating capacity of the station being 20 000 e.h.p.; but the average load carried is 15 000 e.h.p. The current, which is 2 200-volt, 2-phase, 66-cycle, is stepped up to 25 000 volts, 3-phase, by 2 750 kw., single-phase, air-blast transformers, and is delivered over four circuits to Montreal, where it is transformed into the city distribution systems.

The complexity of distribution at Montreal may be appreciated when it is noted that the current is of three different frequencies, namely, at 30 cycles from Shawenegan, at 60 from Lachine, and at 66 from Chambly, to which must be added the ordinary conversions to D. C., and to various distributing voltages, A. C.

The total distribution of power in the city from the various stations of the company varies from, say, 30 000 h.p. in the summer to as much as 53 000 h.p. at peak load in the winter, and this amount will be supplemented soon by delivery from the Soulanges station now under construction.

St. Lawrence River Branches.—The St. Lawrence River Basin is traversed by many large tributaries which will be discussed briefly in respect to such developments as have been carried out, but it may be stated briefly that, although many small towns and mining and milling centers have hydraulic and electric stations of modest proportions satisfying present needs, there yet remains opportunity for developing

millions of horse-power on such rivers as the Ottawa and its tributaries, the St. Maurice, the Saugenay, the Trent, Spanish, Nepigon, etc., and projects innumerable of greater or less merit are being studied carefully by engineers and financiers with a view to meeting industrial needs as they arise, and creating new centers and industries to consume the available power.

The Kaministiqua Power Company has constructed a generating station at Kakabeka Falls, on the Kaministiqua River, and transmits power to the City of Fort William (population 15 000) at the head of Lake Superior, where grain elevators, flour mills, foundries, and municipal utilities consume some 6 000 or 7 000 h.p. at the present time.

The development has a hydraulic head of 175 ft., and carries the water through two 10-ft. concrete tubes a distance of 6 500 ft. to a compensating reservoir, and thence by steel pipes to the power-house. The equipment consists of two 4 000 kw., 3-phase, 60-cycle, 4 000-volt units, driven by German-built turbines, and the current thus generated is raised to 25 000 volts by two banks of single-phase, air-cooled transformers of 1 500 kw. per phase; it is transmitted 19 miles over two circuits of No. 00 copper on wooden poles to Fort William.

In 1904 the Canadian Copper Company (otherwise the International Nickel Company) constructed a generating station on the Spanish River at Turbine, from which current is delivered to its mines and smelters at Copper Cliff near Sudbury, which lies north of Georgian Bay.

The construction of a large head reservoir created good operating conditions, and increased the hydraulic head to 85 ft. Two 2 000-kw. units are installed and a third is now under construction. The current as generated is 3-phase, 25-cycle, at 2 400 volts, and is stepped up to 35 000 volts for transmission. It is carried 29 miles to Copper Cliff, by two 3-phase circuits of No. 1 copper mounted on a double-pole line.

The company has various sub-stations where the current is distributed for service in the mines and at the smelters, which it is understood turn out more than one-half of the world's consumption of nickel.

The Mond Nickel Company is also installing a 1 500-h.p. hydro-electric station for similar purposes on the Vermilion River, and the current is to be transmitted 11 miles to the Victoria mines and smelters, 25 miles southwest of Sudbury.

The Town of Orillia (population 6 000) has constructed a power-station on the Severn River, and transmits some 1 600 to 1 800 h.p., at 22 000 volts, a distance of 20 miles. It distributes the current for industries as well as for pumping and lighting; and, by offering low power rates, has established a stable industrial center.

The City of Ottawa (population 75 000) is in the midst of many developed and undeveloped water-powers, and the latter, in the event of the construction of the Georgian Bay Ship Canal, would become readily available at low investment costs. In addition to the Ottawa River itself, many of its larger branches, especially in the Province of Quebec, are very great latent sources of hydraulic power, and it may be stated conservatively that within a radius of 75 miles of Ottawa City there can be developed on the Ottawa River and its main tributaries 500 000 h.p. at low water, which amount would be increased materially by the construction of the ship canal.

Near or in the City of Ottawa there are ten hydraulic and hydro-electric developments operating under various heads, but chiefly on the Chaudière Falls at from 22 to 35 ft. These developments form the industrial center of the city, and operate very extensive pulp and paper mills on both sides of the river. There are also cement mills, carbide furnaces, and other industries, and power for street railways, lighting, and general city use is also produced. About 35 000 h.p. is now developed, and, as many conflicting interests were contending for their respective shares of the flow of the river, it became evident that some improvement was imperative, so that an agreement to construct a proper dam at Chaudière Falls has been entered into and the work is in progress.

This dam will increase the hydraulic head somewhat, and will drown out the rapids immediately adjoining, thereby lessening the very serious present difficulties in respect to operating in the winter season, when the water is full of frazil, and chiefly to conserve the last drop of water in low-water periods, thereby increasing the available power at such seasons. However, in the near future, more power will be required, and it is proposed that the city itself develop the splendid powers quite near it on the Gatineau River, where a head of more than 80 ft. can be secured at a point within 8 miles of the city, and where 50 000 h.p. can be developed at one location. Considering the vast timber and mineral resources tributary to Ottawa, and its position as

Capital of the Dominion, it may be considered certain that the availability of unlimited hydro-electric power assures the industrial future of the city.

Quebec.—The Ottawa is only one of many great rivers flowing into the St. Lawrence from the north between Montreal and the sea, and, as all these rivers rise in unbroken forests on the Laurentian highlands, and are all well supplied with splendid storage facilities, they are all valuable sources of power. However, the sparse population has been a deterrent, and the only large works north of the St. Lawrence, and devoted to the generation of electric energy for transmission purposes, is on the St. Maurice River at Shawenegan Falls, where the Shawenegan Water and Power Company has created a large industrial town and also generates a large amount of power for transmission to Montreal, Sorel, Three Rivers, Thetford, etc., and will doubtless soon enter the field at Quebec in opposition to the present companies.

The hydraulic conditions at this place are most favorable, as the head-works consist of a natural lake, which prevents trouble from frazil in the winter. The drainage area is some 18 000 sq. miles, and a flow of 10 000 cu. ft. per sec. is considered the minimum. This, at a hydraulic head of 130 ft., provides more than 100 000 e.h.p. at all seasons. The works have been carried on partly for the generation of electric energy and partly to operate local industries hydraulically. The products of these industries are pulp, paper, aluminum, carbide, etc.

The electric station has three 4 000-kw. and two 7 000-kw. units, generating 2-phase, 30-cycle current at 2 200 volts, and this is transformed to 50 000 volts by banks of 1 100-kw. and 2 200-kw. single-phase, water-cooled transformers having a total capacity of 32 000 kw.

The transmission circuits are of aluminum cable on wooden poles. The Montreal service (85 miles) has been in operation for the past 10 or 12 years, and has given good satisfaction.

The delivery in Montreal is by two circuits, and amounts to 16 000 h.p.; besides which, the company serves the asbestos and other mineral districts on the south side of the St. Lawrence at Danville, Thetford, etc. The accomplishments of this company within the last ten years have been considerable; indeed, its works form the basis of many large industries, and it can be confidently expected that the whole 100 000 h.p. available at this one water-power will be made use of in the near future.

While Shawenegan is an instance of the water-power being developed as a foundation for industries and to supply distant markets, the developments in the vicinity of Quebec City (population 90 000) have been made strictly to supply existing needs. The installation on the Chaudière River, south of the St. Lawrence, has a hydraulic head of 114 ft., and is laid out for 6 000 h.p., but the equipment, thus far, is for not more than one-half this amount. The current is 3-phase, 66-cycle, and is generated at 11 000 volts. It is transmitted to Levis and also to Quebec City by insulated cable laid on the bed of the river.

Two developments north of the St. Lawrence, on the Montmorency River and on the Jacques Cartier River, supply current for city lighting, street railway operation, cotton mills, etc.

The Montmorency works, having a capacity of 4 000 h.p., have five units operating under a head of 220 ft., and generate 2-phase current at 5 000 volts, which is transmitted direct over four circuits to the city sub-stations.

The Jacques Cartier works have 2 000 h.p. installed, and the current is transmitted 18 miles at 20 000 volts over two 3-phase circuits to the city limits, where the company has a steam auxiliary station of about 2 000 h.p. to carry peak loads.

Some Conclusions.

A careful consideration of the electrical data contained herein will indicate the trend of activity in Canada in hydro-electric investments and the reasons therefor. In general, it may be said that on the Atlantic Coast and over the western and central portions of the great plains very little benefit is to be derived from such constructions, but over all the Provinces of New Brunswick, Quebec, Ontario, Eastern Manitoba, Western Alberta, and British Columbia, the continued construction of additional hydro-electric stations and the distribution of electric current to the practical limits of transmission may be confidently predicted. The resources of Canada, in minerals, timber, and agriculture, are so varied and so immense that the growth of industry as a legitimate outcome of the development of these resources, and the corresponding increase in population, may be looked on as a foregone conclusion, and it is as certain that, in this industrial growth, low-priced power will form a large factor.

At present the hydro-electric developments serve to a greater or

less extent such industrial centers as Montreal, Quebec, Ottawa, Peterboro, Toronto, Hamilton, Winnipeg, and Vancouver, besides a great number of smaller ones; but only a beginning has been made. Probably the 500 000 h.p. now hydraulically developed would include only stations of 1 000 h.p. or more, and there are literally many millions of horse-power available for development within easy reach of the present population; but, aside from the increased industrial demand, there are specific demands of great importance. Already, as noted, several mining interests are served by hydro-electric power, and, with the growth of mining investments, the importance of such service will increase. Then again, pulp and paper (with the depletion of United States supplies) will become a much more important industry than at present, and consequently much hydraulic and electric power for 24-hour service will be thus consumed.

Particular attention, however, is directed toward the enormous opportunities for electro-chemical industry. At Niagara Falls, N. Y., a large group of such industries is now well established, and in Canada a modest beginning has been made at Shawenegan, Ottawa, and Niagara Falls, Ont.; but Canada has in its great rivers sources of power as cheap as any in the world, except that the labor item may increase slightly the investment per horse-power beyond that of a similar construction in Europe. The whole question rests on transportation and markets; the latter, artifically restricted by the United States tariff, must seek an outlet in Great Britain, and, as it is well demonstrated that $10 per horse-power-year, continuous power, at the low-tension switch-boards, is a remunerative price for large installations, and will cover all fixed charges, depreciation, and attendance, there would appear to be a good field for investment.

The financial condition in Canada at present, as regards hydro-electric enterprises, is worth considering. Some 10 or 15 years ago many of the brighter minds in the country seized the idea that, by securing valuable long-term franchises, on the one hand, for electrical service, and on the other for water privileges on near-by rivers, a splendid opportunity existed for profitable exploiting.

The net result at present is that, in almost all the large towns and cities, these services are administered by companies whose charges for power and light are only regulated by two things: either the exigency of the occasion, or the cost of a similar service produced by the

customer from coal usually imported from the United States; and as a good quality, run-of-mine, bituminous coal costs from $3.50 to $4.00 per ton in Ontario and Quebec and $6.50 per ton in Manitoba, it is evident that such competition is harmless, and the electrical companies were and are able to do business at very profitable rates.

In recent years, however, the municipalities have become active, and several smaller towns and cities have constructed municipal hydro-electric stations. The movement is very active at present in the Province of Ontario, and to a less extent in other provinces.

The struggle is bound to be prolonged and bitter; indeed, much bad feeling and resort to disreputable tactics have already been exhibited. Municipal and provincial credit has been attacked in the London market, and all the influence which heavy vested interests can exert is and will be directed against further municipal action.

The ultimate result is evident; the public mind will become more and more inflamed by the obstruction of the electrical companies, and, undoubtedly, in the end, the municipalities will carry out the expressed wish of the majority; but in the meantime public credit is unsettled, the manufacturers are playing a waiting game, and no one is benefited because the expansion of the market, and consequent advantage to every one, including the electrical manufacturing companies, has not taken place.

It would appear advisable to consider the appointment of a Public Service Commission by the Dominion Government to regulate the rates which may be charged by companies already in operation, so as to secure that benefit to the public which the presence of abundant water-powers warrants, because it is not probable that, at the most, more than a majority of the industrial centers will be served municipally, and, judging by the results obtained by the Dominion Railway Commission, much satisfaction might be obtained and the public mind be set at rest if a rate-regulating commission for electric utilities, with wide powers of action, should be appointed.

Concurrently, however, with the distribution of electric power for utilization in cities, etc., it is to be anticipated that very large constructions will be undertaken in the near future to furnish power for the operation of railways over heavy grades, where coal is expensive or traffic dense, and that at the same time the natural resources of the country will develop to an increasing extent, and that

in mining work, pulp and paper manufacture, electric smelting, and general electro-chemical service will be found the most attractive fields for the enormous and not fully appreciated sources of cheap and widely-distributed power which is now wasting in so many districts of Canada.

DISCUSSION.

Mr. Holgate. H. HOLGATE, M. AM. SOC. C. E. (by letter).—This subject is one of the great features of present-day development and advancement, and is of growing importance and interest to the community.

The well-known phrase "Conservation of Natural Resources" can probably be applied to hydraulic power development along lines similar to those which apply to forests and their preservation or destruction. A forest may be wasted or burned, but the land, in most cases, will reproduce something of value. In developing hydraulic power, a permanent condition of affairs may be created, so that the full potentiality of the river at the development site can never be realized, thus causing a permanent waste of power. Of course, there are commercial features which sometimes limit the capital outlay and prevent the building of works to produce maximum results, but, it often happens that permanent works are built for present requirements only, and are afterward extended or improved to meet the growth of business, generally resulting in a greater outlay than would have been necessary originally, and also, even then, owing to the limits imposed by initial design, failing to be able to utilize all the power which might be available, or even may be required.

No water-power proposition should be undertaken before it has received the fullest possible study; and investors would be consulting their own interests if they secured sufficient information to establish the total physical possibilities, and the cost of the development. It is of first importance to establish the full potentiality of the stream, making use of all favorable physical features; then any modifications which business prudence may dictate may be made, but one should not lose sight of the fact that at a future time it may become necessary to expand and carry out an ideal form of development; therefore, the earlier works should be built to fit in with future work, as far as practicable, and thus avoid unnecessary expense and costly interruption of business.

It is not possible to say what is to be in the future, but lack of confidence in the future progress of Canada has stamped itself on many a hydraulic power development, and the time has come when all doubts and fears for the great expansion of trade in the Dominion may be dismissed; Canada may now be said to have emerged from the "hobbledehoy" stage of growth and to have entered upon good sturdy progresssive manhood, with growing confidence in the future.

In granting water-power rights, it will be proper for the Government to insist that the initial works shall be built with due regard to the possible maximum development in the future, so that all possible power can be made available when conditions arise which may demand its use.

Mr. Holgate.

Though Mr. Smith's paper does not contemplate going into details, yet he has mentioned two plants which were designed and built by the writer and are used extensively for mining and smelting purposes. The first is the development at Bonnington Falls, on the Kootenay River, in British Columbia, and the second is the development for the International Nickel Company at High Falls, on the Spanish River. The latter serves all the works of the International Nickel Company in what is known as the Sudbury District, in Ontario.

As these two plants are radically different in design, and represent different methods of development, their cross-sections, Figs. 6 and 7, may prove interesting.

The water-powers of Canada are distributed over almost the whole country, but the district east of Port Arthur is particularly favored, and, owing to the accessibility of the powers south of the height of land, those on rivers tributary to the St. Lawrance will naturally be developed before those on rivers flowing into Hudson Bay (erroneously called Hudson Sea).

In particular, attention is called to the possibilities of the St. Lawrence, and to the rivers tributary to it east of Quebec, as this territory, in so far as the north shore of the St. Lawrence is concerned, is not referred to in Mr. Smith's paper, nor has it received much attention from any one.

In regard to the St. Lawrence River, the mean level of Lake Ontario is 245 and the mean level of the St. Lawrence at Montreal is 18, thus creating a fall of about 227 ft. The minimum flow of the river is about 187 000 cu. ft. per sec., so that, on this basis, the minimum energy of the river in this stretch in about 100 miles is about 3 500 000 h.p.

The fall in the river is overcome by a canal system affording a depth of 14 ft., with locks 255 by 45 ft. The various canals are:

Canal.	Lockage.	Length.
Galops	17.81 ft.	7.60 miles.
Morrisburg	14.81 "	3.70 "
Farren's Point	10.78 "	0.75 "
Cornwall	46.82 "	11.50 "
Soulanges	84.13 "	14.00 "
Lachine	47.62 "	8.50 "
Totals	221.97 ft.	46.05 miles.

This leaves stretches of river navigation aggregating about 54 miles, so that nearly one-half of the navigation distance from Lake Ontario to Montreal is through the canals.

The present depth of the canals will not carry the larger Lake boats, and the question of deepening the canals and enlarging the

Mr. Holgate. locks is being discussed. This would be a very heavy undertaking, and would result practically in rebuilding all the canal works, as the new depth should be at least 22 ft.

An exhaustive investigation into the feasibility of damming the St. Lawrence at the foot of the Long Sault Rapids, a few miles west of Cornwall, has been carried on by J. W. Rickey, M. Am. Soc. C. E., during the past three years, and the writer has examined all the data gathered, and has no doubt of the possibility of carrying out the work. The result of building dams at the sites selected will be the total elimination of floods above the rapids, caused now by ice jams, and the improvement of flood conditions due to ice below the rapids, and, in addition to this, the present Cornwall Canal could be eliminated by placing a single lock at the dam.

Ice conditions on the St. Lawrence are annually a source of trouble, and, of course, this would be prevented by treating the rapids in this manner. The writer is convinced that a careful study will show that it is possible to dam the river at the foot of each of the rapids, thus providing for navigation throughout the whole distance, and using the present canal system as an alternative route. In doing this, the necessary depth could be obtained, and navigation conditions would be decidedly improved by providing a deeper channel and by permitting vessels to move with increased speed, as they must now proceed through long stretches of canal with numerous small lockages, making their progress very slow, both up and down.

Of course, the tourist will complain that he cannot "shoot the rapids," and that the natural beauty of the river is marred, but when it is considered that carrying out such a work will provide a vast amount of power, and also will improve a great international water highway, the argument of the lover of wild Nature can have little weight.

In some cases the construction of such works would not be justified if the result were only the production of power, but coupled with this there is the possible improvement to navigation, and the two results will justify at least an exhaustive study of the river, with this end in view; particularly is this so when the increasing value of power is considered, and also the increasing importance of this water route.

The relations between the Nations are happily such as to render it possible to bring about a solution of the problem, and if they co-operate, the St. Lawrence may be made a gigantic power producer, and a more important water highway than it is now.

Attention is next drawn to the possibilities offered for the development of water-power east of Quebec, on the rivers flowing south into the St. Lawrence. These begin with the Saguenay and may be said to end with the Natisquahan River, although there are gigantic water-powers on the Hamilton River, which, at present, are commercially inaccessible.

Mr. Holgate.

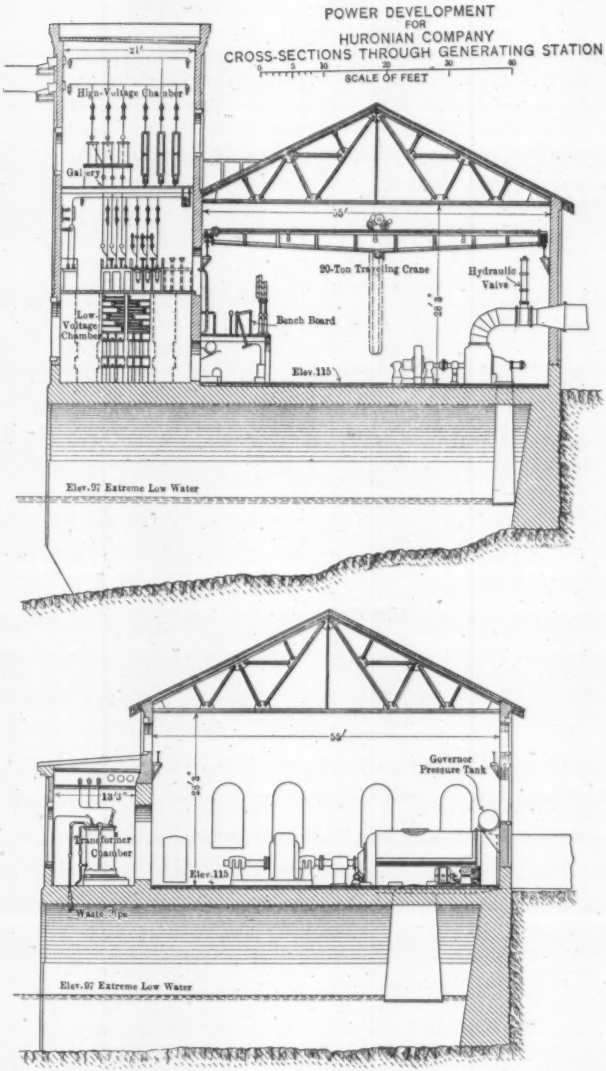


FIG. 6.

Mr. Holgate. Large developments are possible on the Saguenay, Bersemis, Godbout, Outarde, Manicouagan, Manitou, Romaine, St. John, and Natisquahan Rivers. A development of power on St. Marguerite River, made by the writer, is used for the manufacture of wood pulp.

An unique situation occurs about 200 miles east of Quebec, where the Outarde and Manicouagan Rivers enter the St. Lawrence about 5 miles apart. There, within a radius of 5 miles, the writer has examined three possible developments, the combined horse-power of which would be 600 000.

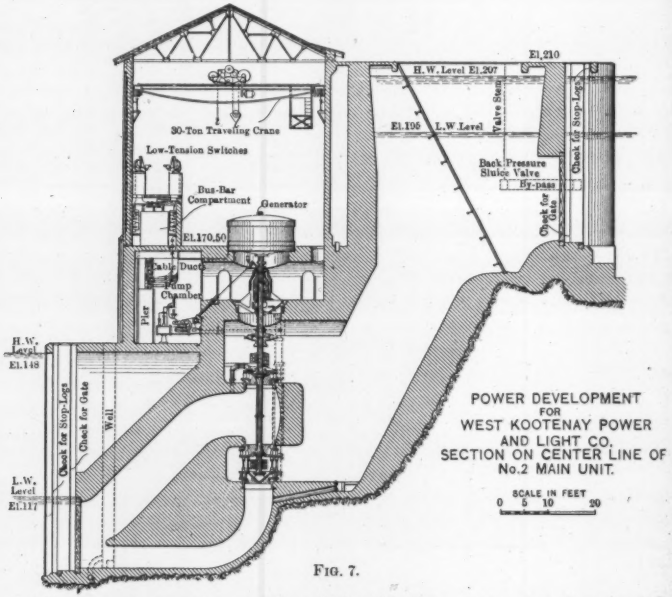
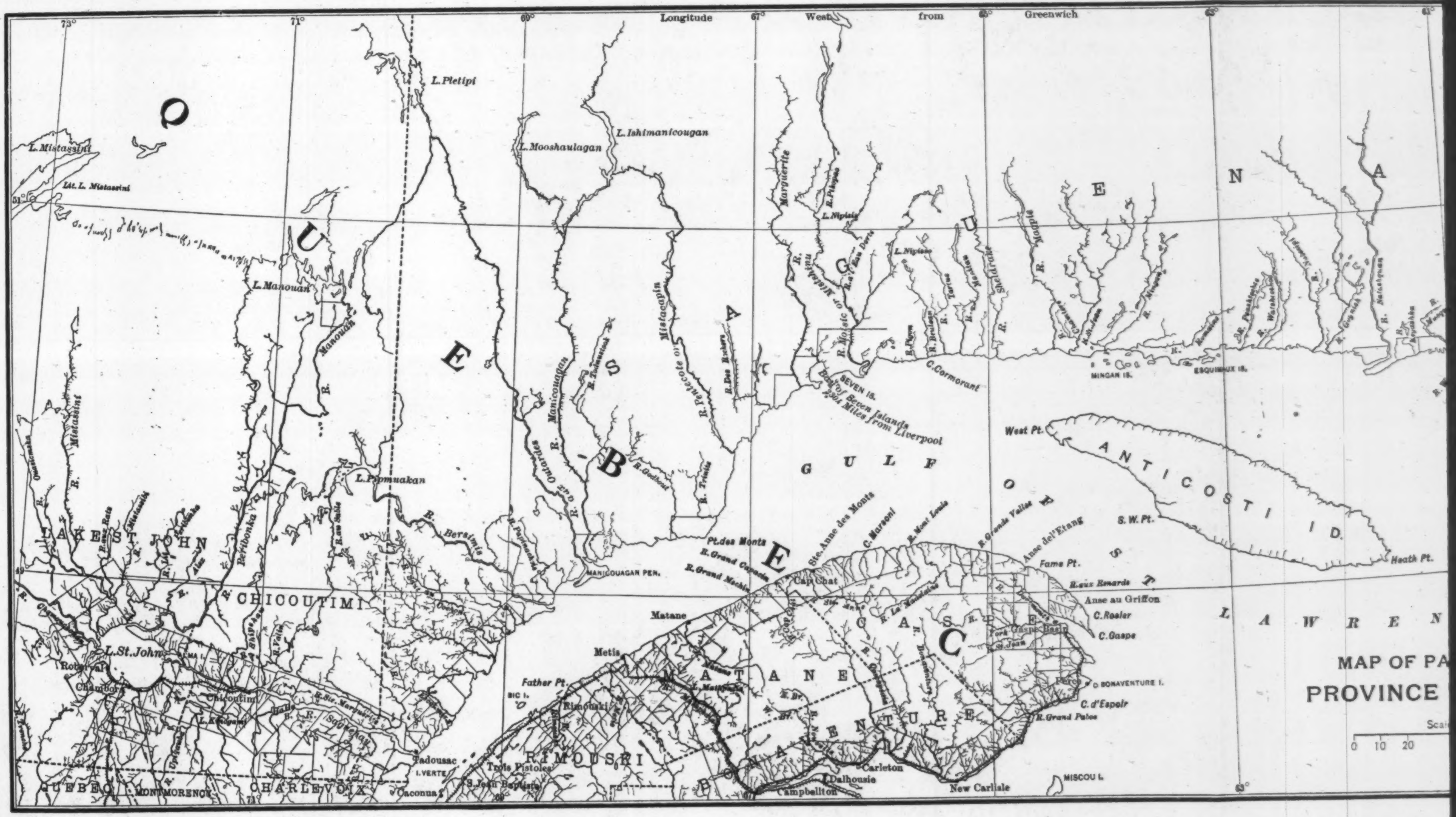


FIG. 7.

All the water-powers north of the St. Lawrence are close to tide water, the back country for a long distance is covered with spruce, and iron exists in large bodies, so that it is only a matter of time when these powers will be commercially utilized. When smelting by electricity becomes a pronounced possibility, these water-powers will become particularly useful; at present they offer inducement principally to the paper-making industry.

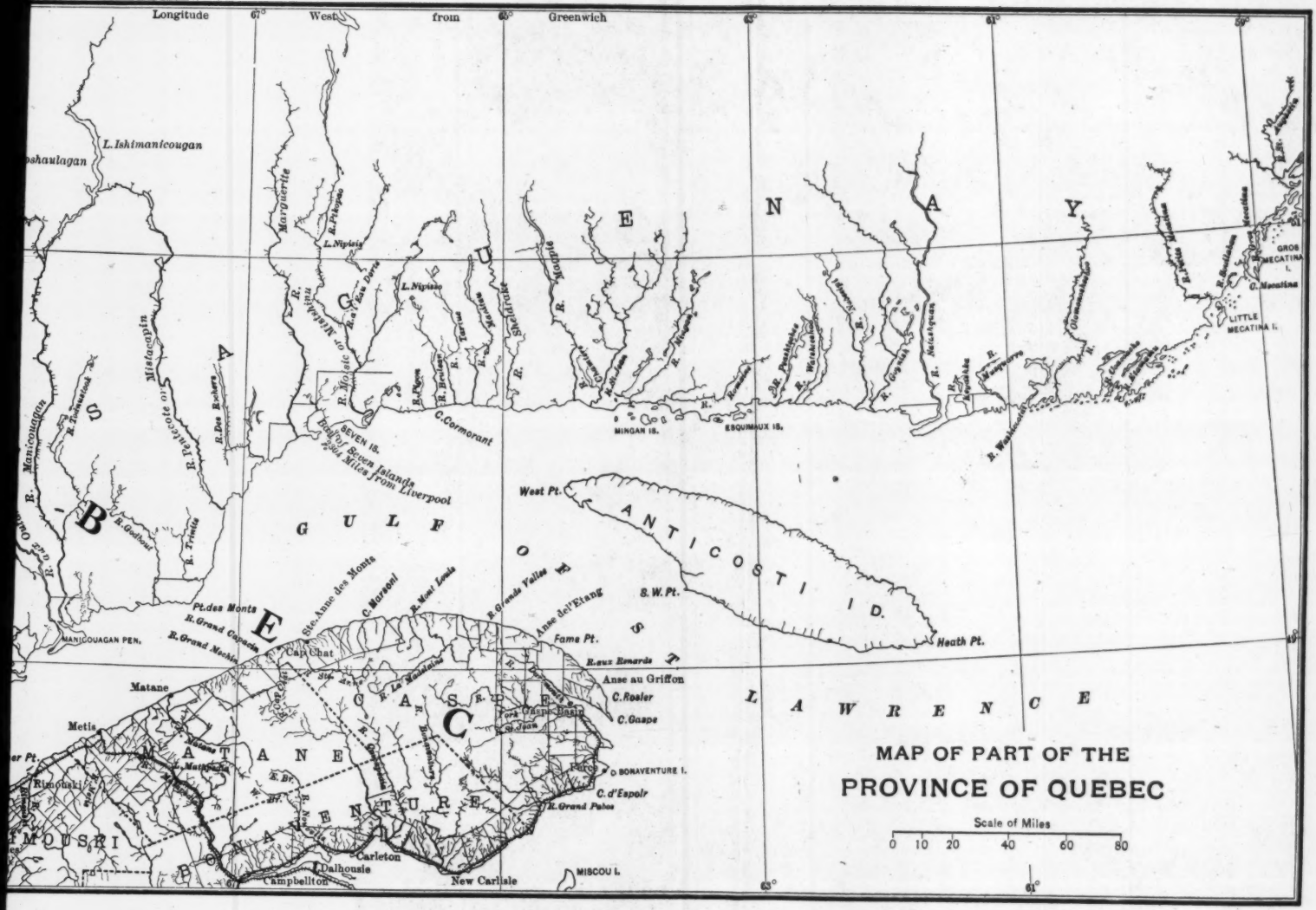
Winter conditions, of course, are severe in this district, but not so severe as to affect operation materially, and the larger the river, the less trouble will be experienced in the operation of the power.

Plate XI, a map of this district, shows the location of the rivers,



Longitude West from Greenwich
73°
71°
69°
67°
65°
63°
61°
51°
49°
QUEBEC
L. Pletipi
L. Ishimanicougan
L. Mooshaulagan
L. Mistassini
Lit. L. Mistassini
L. Manouan
Manouan
L. Pipmuakan
R. Bersimis
Mistassini
Ouasiemska
LAKE ST. JOHN
CHICOUTIMI
L. St. John
Roberval
Chambord
Chicoutimi
Peribonka
R. Saguenay
R. Ste. Marguerite
L. Kenogami
Tadousac
I. VERTE
Trois Pistoles
Cacouna
S. Jean Baptiste
QUEBEC
MONTMORENCY
CHARLEVOIX
R. Outardes
R. Manicouagan
MANICOUAGAN PEN.
R. Godbout
R. Trinite
R. Pentecote or Mistacapin
R. Des Rochers
Marguerite
R. Pikopao
L. Nipisis
L. Nipisso
R. Moisic or Mistashinu
SEVEN IS.
Bay of Seven Islands
2304 Miles from Liverpool
C. Cormorant
Sheldrake
R. Magpie
R. St. Jean
MINGAN IS.
ESQUIMAUX IS.
R. Natashquan
GULF OF ST. LAWRENCE
West Pt.
ANTICOSTI ID.
S. W. Pt.
Heath Pt.
Pt. des Monts
R. Grand Capacin
R. Grand Mechin
Ste. Anne des Monts
C. Marsoni
R. Mont Louis
R. Grande Vallee
Anse del'Etang
Fame Pt.
R. aux Renards
Anse au Griffon
C. Rosier
C. Gaspe
York
Gaspe Basin
St. Jean
Perce
BONAVENTURE I.
C. d'Espoir
R. Grand Pabos
Matane
Metis
Father Pt.
BIC I.
Rimouski
RIMOUSKI
MATANE
GASPE
BONAVENTURE
Cap Chat
Carleton
Dalhousie
Campbellton
New Carlisle
MISCOU I.
MAP OF PA
PROVINCE
Scal
0 10 20

PLATE XI.
TRANS., AM. SOC. CIV. ENGRS.
VOL. LXV, NO. 1123.
HOLGATE ON
HYDRO-ELECTRIC POWER IN CANADA.



and it should be noted that the distance from these points to Europe is much less than from any of the Atlantic ports. Mr. Holgate.

The water-power possibilities of Canada are widespread and very great, and under wise direction, with the rapid growth of the country, it is reasonable to expect a great advance in the utilization and development of hydraulic power. Though many sections of the country where important developments are in operation have not been mentioned, it will be seen that much has yet to be done before the resources of Nature are even partially made use of.

Electro-chemistry, requiring large quantities of power at low cost, will find a great field in the water-powers of Canada, and will no doubt assist in the early advancement of power development.

E. J. BEUGLER, M. AM. SOC. C. E. (by letter).—Mr. Smith's paper forms a comprehensive account of the development of Canadian water-powers. Mr. Beugler.

This development appears to have followed the natural laws of supply and demand, and is in accordance with the general policy that water-power plants are constructed only when the power can be generated profitably, and not before, unless the project is developed prematurely. The latter condition, unfortunately, is the case with several developments in the United States which, thus far, have failed to pay dividends.

In districts where cheap coal or other fuel can be secured for the generation of power, the hydraulic plant, unless very favorably situated as regards cost of construction, cannot be developed profitably. Many water-power projects that could not now be operated to advantage will ultimately prove valuable. It may be news to some to know that the first cost of a complete hydro-electric development, and consequently the fixed charges or interest on this cost, is, as a rule, greater per horse power generated and delivered than the same elements in a steam plant.

It is also of interest, in view of the general disposition toward preserving "water-power resources" in the United States, to consider the fallacy of certain ideas regarding conservation. This matter has been taken by many to mean that our water-powers, coal beds, and forests should be saved for future generations at the expense of present needs and economies. To the writer's mind this view is entirely wrong. Our resources are for our judicious use whenever demanded, and while wanton waste is also wrong, the policy of discouraging capital from investing in water-power developments by threats of onerous taxes, laws, and regulations simply results in a loss to the public from the delayed use of these resources, some of which, if not used immediately, are irrevocably lost.

The construction of water-power plants will not economize our supply of coal so long as the coal can be bought for a price which will produce a certain amount of power for less money than that for which

Mr. Beugler. it can be produced in a water-power plant, nor would it be sound business policy to develop such plants under this condition.

Water-power plants should not be subject to extraordinary government taxes. Taxing a water-power company for the use of water, where the company has already paid a good price for riparian rights and necessary lands, simply shifts the line of tax payment. The power company must add this item to its operating costs, and consequently must sell its power for a higher figure, so much higher, in many cases, that the water plant would be unable to compete with coal power; hence, any additional tax upon the development of water-powers may have the effect of preventing such development and cause a continued use of a failing supply of coal, an effect which is diametrically opposite to that desired by the devotees of conservation.

Subsidy with governmental regulation, the latter as advocated by Mr. Smith, would appear to be more logical than additional taxation.

The point the author makes regarding favorable topographical conditions is an important one, especially as to the beneficial effect of swamp and lake areas on minimum stream flow. There are anomalies, however, which give opposite results from what might be expected and cannot be explained satisfactorily by the difference in the rainfall and weather conditions. For example, a comparison of the minimum run-off (as recorded by the United States Geological Survey) for New England streams and for those of the southern Appalachian district shows that for the northern district, with many lakes, the minimum ranges from 0.2 to 0.4 cu. ft. per sec. per sq. mile on water-sheds of from 1 000 to 2 000 sq. miles, while in streams of the southern district, with practically no lakes or swamps, it ranges from 0.4 to 0.6 cu. ft. per sec. per sq. mile for water-sheds of the same area. The comparison is made for equal areas of water-shed, as the general law appears to be that the amount of run-off is in an inverse ratio to the water-shed area, that is, the smaller the water-shed, the greater the minimum run-off per square mile to be expected for any particular stream.

Mr. Smith. CECIL B. SMITH, M. AM. SOC. C. E. (by letter).—Mr. Holgate's discussion, suggestive, as it is, of the widespread and extensive latent hydraulic powers capable of development in various parts of Canada, might do well for the subject of future papers, as even the Engineering Profession itself is not well informed on this subject; the writer's intention, however, was to outline broadly the present achievement.

In Sweden and Norway a large block of power is devoted to nitrogen fixation and other electro-chemical uses, and one may look confidently to the same action as regards some of the larger available powers on the St. Lawrence and Atlantic seaboard; in the interior, however, one may expect that industrial demand, keeping pace with population and manufactures, will, as in the past, be the determining feature. Too much attention, however, cannot be given to those impediments, partly physical, partly political, which have ended in many

constructions being financial failures. The physical obstacles are indicated by Mr. Holgate, in that the design which will give the best future results is often attempted prematurely, making the fixed investment too large for the revenue obtainable during the first few years after operation has commenced; or, on the other hand, very frequently cheap temporary construction is adopted for the purpose of lessening the capital cost, with the result that just as the plant begins to secure a large market, the works must be rebuilt in order to meet the current demands. Mr. Smith.

It is a common fallacy, which finds expression in the public mind, that power companies are making excessive profits, but the public does not fully recognize that a heavy original investment must often wait for profits and take the risk.

The logical deduction is that where, as in Canada, the water-powers are still largely vested in the Crown (or State), the wise policy, looking to the best industrial interests of the country, would be to grant power rights to responsible parties on very generous terms, but with a strict requirement as to actual use, and a tight rein as to rate regulation by a commission appointed for the purpose. As long as the investing public retained confidence in the fairmindedness of such a federal court or commission, there would be every encouragement to develop on liberal lines, and this is the present policy of the Dominion Government. Mr. Beugler goes a step farther and suggests subsidy, and, as there is a strong demand on the part of the public for electric power at low rates, this is hardly compatible with speculative risk and long shots for large future profits, or, in other words, large stock issues in proportion to the bonds.

Canada has had many years of subsidy land grant, and now has a bond guaranty for railways, and might it not be a fair balance of interests if a city, province, or state, or even the Federal Government, would, for large undertakings involving heavy initial outlays, guarantee the bond interest, thereby lowering the rate of interest and doing away with the excuse of large stock issues.

Combining bond guaranty with rate regulation would place hydro-electric investments on a sound low-rate basis, thereby assuring the public of not only a low rate for light and power, but a steadily decreasing rate as the volume of business increased.

From residence in the Southern Appalachian districts the writer would conclude that the minimum flows were much less than those quoted by Mr. Beugler, in fact, there is good authority for a minimum on these rivers of less than 0.1 cu. ft. per sec. per sq. mile of water-shed. As regards area of water-shed, the writer's experience has been that the smaller the area, the less the minimum run-off per square mile. This accords also with the law of probability, as a small water-shed would be much more apt to suffer from an acute drought than a large one.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1124

TESTS OF BUILT-UP STEEL AND WROUGHT-IRON COMPRESSION PIECES.*

By Arthur N. Talbot, M. Am. Soc. C. E., and Herbert F. Moore, Esq.

With Discussion by Messrs. John C. Moses, and Arthur N. Talbot and Herbert F. Moore.

The principal tests described in this paper were made on the following compression pieces: (*a*) a steel column (called Column No. 1) built up of angles, plates, and lattice bars, all the parts being light; (*b*) four wrought-iron bridge posts which had seen long service in a bridge truss; and (*c*) three posts and a top chord in a railroad bridge under service. The tests of (*a*) and (*b*) were made in a testing machine; for (*c*) a locomotive and cars formed the load. Auxiliary tests were made on lattice bars and other parts.

These tests were taken up with a view of determining experimentally: (1) something of the variation of stress, both longitudinal and lateral, throughout the channel members of the compression piece; (2) something of the amount and distribution of stress in the lattice bars of columns, and the action of similar bars under separate test; and (3) the general relation between the component parts and the column as a whole. The slenderness ratio of the columns, $\frac{l}{r}$, ranged from 38 to 66. It will be seen that, instead of following the more

* Presented at the meeting of June 2d, 1909.

common effort to determine the effect of length of column, emphasis was placed on measuring the distribution and range of stress over the various parts of the column.

It is unnecessary to state that built-up compression pieces (whether long or short) are not perfect, the natural imperfections of the component parts being increased in the process of fabrication. To non-homogeneity of structure and lack of straightness in the component angle or channel are added such further imperfections as kinks and eccentric connection of parts, which go to increase the opportunities for local flexure in the component parts and for flexural stresses in the column as a whole. An attempt has been made in these tests to measure the deformations in the presence of such conditions, and to find the general distribution of stress. In view of the many limitations surrounding such tests, the results are to be taken as suggestive and qualitative, and not as exact determinations.

Results of Tests.

The following observations on the results are given here with a view of showing the trend of the tests:

I. Variation of Stress Throughout the Length and the Cross-Section of the Channels and of the Column as a Whole.—Channel members show evidence of considerable local flexural action, such as may be produced by lack of straightness or by any method of applying the load eccentrically. This is especially true in the flimsier column.

The condition of flexure varies markedly throughout the length of the member, in some cases the maximum compression in one cross-section being at the extreme fiber on one side of the channel, and in a near-by section the other side of the channel showing the excess of stress.

There were indications of sudden changes in the relative amount of stress carried by the two channels at near-by sections, indicating general flexure of the column.

The measurements made indicate in a number of cases stresses in the extreme fiber from 40 to 50% in excess of the average stress, and in some cases even higher.

The amount of eccentricity necessary to account for the increase of stress found in individual channels, based on lack of straightness and the ordinary theory of flexure, is relatively small.

II. Stress in Lattice Bars.—The amount of deformation observed in lattice bars is relatively small.

The measurements indicate a stress in the lattice bars which would be produced by a transverse shear equal in amount to 1 to 3% of the applied compression load, or to that produced by a concentrated transverse load at the middle of the column length equal to 2 to 6% of the compression load.

III. Tests of Lattice Bars.—Tests of lattice bars for load-carrying capacity show that the usual form of bar is a very inefficient compression member when loaded eccentrically through a riveted connection.

Under common conditions of loading, the maximum fiber stress may be as much as three times the average stress.

In the compression tests of single lattice bars, the ultimate strength was in no case as much as one-half of the elastic limit of the material.

IV. Relation between Component Parts and the Column as a Whole.—It is a question whether the component members of a built-up column act together to form an integral compression piece, especially in resisting oblique distortion.

The distribution of stress under working loads, and even up to incipient failure, may be quite different from that which exists when the column becomes crippled. This is due to the yielding of the more strained parts after the yield point is reached at any fiber, and the consequent redistribution of stress.

No relation has been found between the stresses actually observed and the stresses computed by column formulas.

The sudden failure of a test column at a relatively low load by buckling of lattice bars is accounted for when the amount of transverse shear developed in other test columns and the strength of lacing found in lattice-bar tests are taken into consideration.

Laboratory Tests of Columns.

Description of Columns.—One steel column and four wrought-iron columns were tested. The steel column was built especially for the purpose of this test by the American Bridge Company at the Lassig plant. The wrought-iron columns were halves of bridge posts taken from an old bridge of the Chicago, Burlington, and Quincy Railroad.

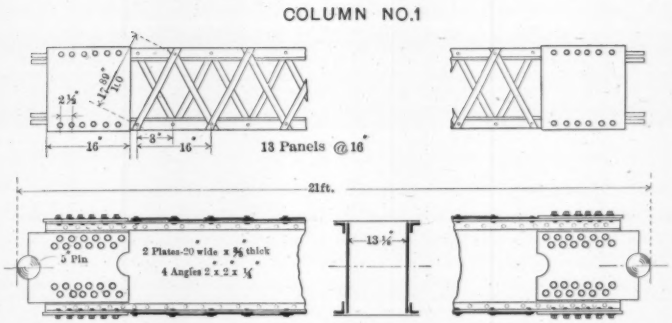


FIG. 1.

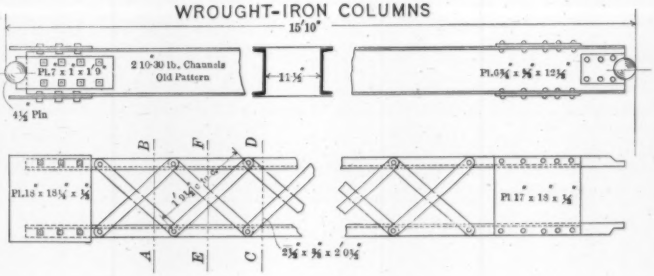


FIG. 2.

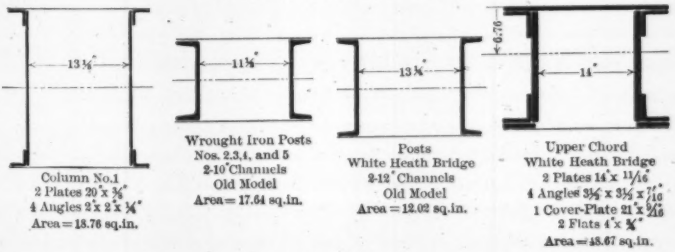


FIG. 3.

TABLE 1.—Data of Columns.

Column designation.	Column No. 1.	Wrought-Iron Posts.		White Heath Bridge.	
		Nos. 2, 3, 4, and 5.	Retests 2*a* and 4*a*.	Posts $U_2 L_2$ N., $U_3 L_3$ S.	Upper chord $U_3 U_4$ scuth.
Area of column section, in inches	18.76	17.64	17.64	12.02	48.67
Length, center to center	21 ft.	15 ft. 10 in	14 ft. 7 in.	25 ft.	19 ft. 10 in.
End conditions	Pin parallel to lacing.	Pin parallel to lacing.	Pin parallel to lacing.	Lower end pin, upper end riveted	Riveted.
$\frac{l}{r}$, axis parallel to lacing	37.8	43.5	40.1	66.1	40.7
$\frac{l}{r}$, axis perpendicular to lacing.	37.2	41.2	38.0	41.0	29.6
$\frac{l}{r}$ of each flange member, axis perpendicular to lacing for full length of column	593	400	367	416	.....
$\frac{l}{r}$ of each flange member, axis as before, for distance between adjacent lacing rivets.	37.7	33.7	33.7	22.2	.....
Lattice bars, dimensions of section, in inches	1 by $\frac{1}{4}$ in. and $1\frac{1}{2}$ by $\frac{7}{16}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in.	$2\frac{1}{4}$ by $\frac{3}{8}$ in.	$2\frac{1}{2}$ by $\frac{3}{8}$ in., on bottom.
Kind of lacing	Single.	Double.	Double.	Double, riveted at crossing.	One cover-plate.
Angle of lattice bar with axis of column	63° 30′	45°	45°	45°	45° on bottom.

Table 1 gives the dimensions of the columns. The details are shown in Figs. 1 and 2; and Fig. 3 shows the cross-sections. The steel column (Column No. 1) was designed with relatively thin component parts and relatively large outside cross-section dimensions. It will be seen that this column is not so stocky as the usual bridge column. This section was chosen because it seemed to offer better opportunities under the conditions of the test for the study of the distribution of stress, longitudinal and lateral, and especially of the stresses developed in the latticing, than a less flimsy column. In the earlier tests, the lattice bars of Column No. 1 were fastened in place with turned bolts in reamed holes so that one set of bars could easily be replaced by another. After several tests were made with the lacing thus fastened, the bars were riveted in place for the later tests. The turned bolts in reamed holes apparently held the column together in the column tests as well as did the rivets. The proportions of the wrought-iron bridge columns were in no way unusual, and the posts represented good practice at the date of construction of the bridge. The columns

became available through the replacing of the bridge with a heavier structure; they were apparently in good condition.

All the columns were pin-ended, the pin being parallel to the plane of the lacing. The details of the ends are shown in Figs. 1 and 2. The wrought-iron posts were cut in two. One end was left as used in the bridge, and bearing plates and batten plates were bolted to the other end.

Procedure of Tests.—The method used in studying the distribution of stress was to measure at various parts of the column, by means of extensometers, the deformation over short distances, and its variation over a section perpendicular to the axis of the column. The extensometers were placed outside of the flanges of one of the channels in such a way that the shortening at any point of the cross-section of the member could be determined on the hypothesis that in this part of the channel a plane section before loading remained a plane section after the load was applied. It is seen that this hypothesis is not dependent upon the integrity of the column as a whole. A similar arrangement was made in observing deformations in lattice bars. The position of the instruments is shown by Fig. 1, Plate XII. Two sets of instruments were used, and sometimes more. Generally, they were placed on the front and back flanges of one channel and then on the other, but sometimes on the front or back flanges of two channels at the same time. This procedure necessitated the removal of the instruments to new positions after several applications of the load, and, in the complete test of any column, involved several hundred applications of the load.

In all the tests to determine the distribution of deformations, care was taken not to exceed the elastic limit at any point. Under these conditions, it is considered, for the purposes of the discussion, that the stress developed at any point is proportional to the deformation. The ratio of the intensity of the stress to the deformation per unit of length, therefore, will be taken to be equal to the modulus of elasticity of the material.

Extensometers.—In the observations of deformation in the channel members of the columns, the extensometers used in most of the tests were Ames test gauges mounted on suitable frames. These gauges were graduated to $\frac{1}{1000}$ in., and were read by estimation to $\frac{1}{10000}$ in. These instruments magnify the change of length by means of a clockwork device rotating a hand on a dial.

Fig. 4 shows the method of attachment of these extensometers to a column. The gauged length was usually 8 or 9 in. As will be seen, the extensometers measured the change of length between points slightly outside of the members tested. From the actual instrument readings were computed the deformations and stresses at the extreme fibers of the flanges of the channels by the ordinary method, which assumed a rectilinear distribution of deformation and stress.

The type of instrument used is light, simple, and convenient, and, under very severe usage in other tests, it had shown itself to be durable. The limits of accuracy of the instrument were fairly well determined by careful calibration. While the maximum errors possible are large, it gives fairly trustworthy results under careful manipulation, and any available instrument of greater precision would be too bulky or too likely to have its parts deranged under the conditions of column tests, especially in field tests, to be entirely satisfactory.

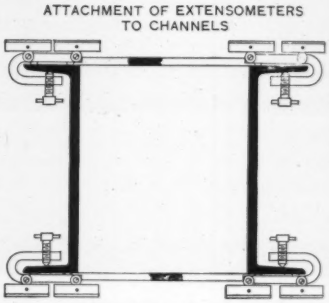


FIG. 4.

The accuracy of all these instruments was tested by comparison with a Brown and Sharpe micrometer acting through a 10 to 1 lever. Basing judgment on the maximum deviation observed in the calibration, and on the smallest deformations measured in any test of columns, it is felt that the error in stress determination for the channel members is generally less than $\pm$ 10%, and that it is well within this limit for most of the determinations in the laboratory tests. This general limit of accuracy is corroborated by a comparison of the average stresses on the center of gravity of the flange members with each other and with the known average stress. To those accustomed to greater precision in calculations and to the greater refinement of most laboratory tests, this limit of accuracy may seem crude. Having in view that the purpose of the tests was qualitative rather than quantitative, and considering the great variation in the distribution of stress found in the columns, and the general consistency of the results, it may be considered that the instruments were satisfactory.

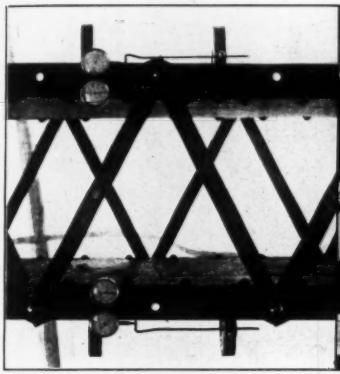


FIG. 1.—EXTENSOMETERS IN PLACE.

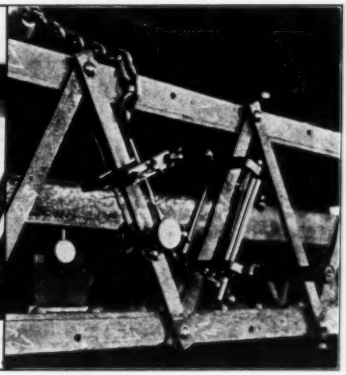


FIG. 2.—ATTACHMENT OF INSTRUMENTS IN CROSS-BENDING TEST OF COLUMN NO. 1.

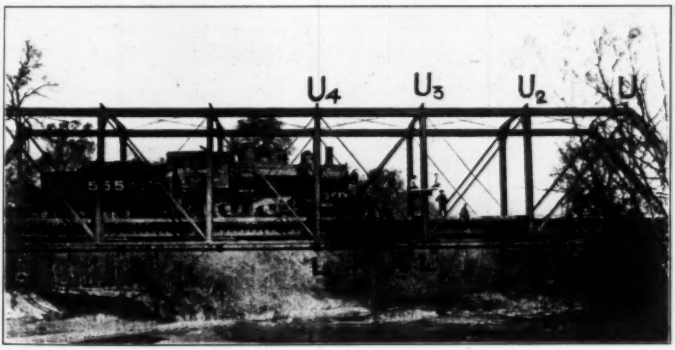


FIG. 3.—WHITE HEATH BRIDGE AND TEST TRAIN.

In several of the tests, both in the laboratory and in the field, extensometers of the Johnson type were also used, but it was found that the results were not trustworthy, and no account of these measurements is given here.

The Ames test gauges were also used to measure deformations in the lattice bars of all columns but No. 1. On account of the very low stresses in the lattice bars, it is felt that the stresses determined in them in this way may be in error by ± 20%, or even more. In Column No. 1, a Ewing extensometer, reading to $\frac{1}{50\,000}$ in. was used, and the stresses in the lattice bars of this column are judged to have been determined with an error in any case of not more than ± 10 per cent.

Testing Machine.—The testing machine used was the Riehle, 600 000-lb., vertical, screw-power testing machine in the Laboratory of Applied Mechanics of the University of Illinois. This machine has a clear space of 36 in. between screws, and will take in compression specimens 25 ft. long. The speed of head used in these tests was in nearly all cases 0.4 in. per minute. This machine is equipped with massive guide frames which take the very heavy side thrust which occurs on the cross-head when a specimen is tested under oblique load. These guide frames are entirely independent of the weighing mechanism.

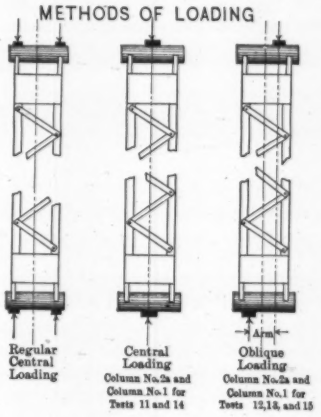


FIG. 5.

Method of Loading.—Two methods of loading were used, central and oblique. In all cases the load was applied at the ends of the column through the pins, and in a plane at right angles to the web and passing through the nominal axis of the column. In central loading, the pin was adjusted to an even bearing on the machine, the object being to apply the load equally to the two webs of the column. In oblique loading, the pin was supported on a narrow block, as shown in Fig. 5, in such a way as to secure a given eccentricity. The center of the block was taken as the point of application of the load. This is approximately true, the error probably not being greater than

¼ in. In two tests with Column No. 1, the point of application of the load is uncertain, as by an oversight the two bearing pieces used on each pin were placed unsymmetrically with respect to the axis of the column. It seems probable that the loading was nearly central.

Routine of Tests for Stress Distribution.—In a test for stress distribution, the column was placed in the machine and a light initial load was applied. The extensometers were then attached in position to measure the deformation at some point of the column, and an initial reading was taken. A known load was applied by the testing machine and the instruments were read again. The load was then released to its initial value and another application of the load was made. If the second readings did not exactly check the first, further applications of the load were made. In cases where the observed deformations were large or seemingly abnormal, the test was repeated at another time, and in some cases as many as ten observations were made on the same gauged length. In some of these cases the instruments were reset, their places being exchanged. The instruments were next attached in a new location, and the process was repeated. Thus the stress distribution in various parts of the column was finally determined. The load generally used in the laboratory tests was 10 000 lb. per sq. in. of section of the column in excess of the initial load.

Results of Tests for Stress Distribution in Channels.—Tables 2 and 3 give results of the tests to determine stress distribution and variation in the flange members found in twelve of the column tests. The stresses given are calculated from the observed deformation, using for the modulus of elasticity 28 000 000 lb. per sq. in. for steel and 26 000 000 lb. per sq. in. for wrought iron, these values checking closely with the total shortening of the columns and with the average deformations observed throughout their length. As heretofore described, the stress noted is the average over a space of 4 or 4½ in. on either side of the point indicated. Any lack of agreement between the average stress on the center of gravity of the flange members and the average stress for the load applied is probably due principally to instrumental errors.

Figs. 6, 7, and 8 show graphically the stress distribution and variation. The full line gives the stress at the east side (front) and the dotted line at the west side (back).

TABLE 2.—STRESSES IN COLUMN No. 1.

	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Test No. 1.												
1	8 100	10 200	10 600	12 200	10 000	9 700	6 500	8 400	8 600	9 600	10 900	11 300
2	9 700	10 100	10 100	8 100	9 600	10 000	11 900	11 500	11 600	8 900	10 500	11 100
3	7 500	10 200	11 000	11 800	10 000	9 700	11 500	9 800	9 800	9 000	10 300	11 000
4	10 500	10 000	10 000	10 500	11 000	11 000	9 000	8 600	8 600	10 800	10 100	10 500
5	9 600	9 100	9 100	10 900	9 700	9 500	10 300	8 500	9 700	8 800	10 100	10 600
6	9 900	11 000	11 100	10 000	9 600	9 700	9 400	8 400	8 100	9 000	9 200	9 100
7	10 000	7 600	6 900	11 000	10 000	9 900	9 000	9 400	9 900	9 200	10 000	10 400
8	8 100	8 900	9 000	10 800	11 300	11 500	12 700	10 500	10 400	12 200	10 100	10 300
9	10 000	11 000	11 100	9 400	11 000	11 000	14 500	10 000	9 000	9 000	9 200	9 200
10	14 000	10 000	9 300	14 300	11 800	11 000	5 400	8 900	9 300	7 200	9 600	10 000
11	8 600	9 500	9 600	10 000	9 800	9 600	9 000	8 800	8 400	9 800	11 000	11 400
12	12 300	9 200	8 700	9 100	9 400	9 800	10 800	9 300	9 000	10 700	9 600	9 500
13	.....	.....	.....	8 700	8 800	9 000	7 800	9 000	9 200	.....	.....	.....
Test No. 2.												
6	11 500	12 100	12 100	11 500	10 000	9 500	9 100	9 000	9 300	5 800	7 500	7 700
7	12 200	12 600	12 700	14 600	13 500	13 300	5 900	7 500	8 000	8 300	9 500	9 500
8	11 300	12 500	12 800	12 400	12 500	10 600	7 800	8 200	8 300	7 500	10 300	10 700
9	14 800	13 300	12 700	9 500	10 500	10 600	11 800	7 300	7 000	10 300	8 400	8 300
10	9 000	10 200	10 400	16 200	10 500	9 600	7 800	10 500	11 400	9 000	8 400	8 500
11	15 200	10 200	9 700	13 700	11 600	11 500	7 600	8 100	8 300	5 000	6 300	8 600
12	10 300	11 900	12 300	12 700	11 600	11 500	9 400	8 200	7 700	7 000	7 500	9 700
13	13 200	11 800	11 700	9 900	10 500	10 800	7 500	8 600	9 100	6 300	6 300	8 400
Test No. 3.												
1	.....	.....	.....	13 800	10 400	9 500	7 500	8 800	8 900	.....	.....	.....
2	8 900	10 600	10 900	8 800	10 400	10 600	8 300	9 300	9 400	10 000	9 700	9 700
3	10 500	9 800	9 800	8 700	9 000	9 200	10 400	8 700	8 700	11 900	10 500	10 300
4	9 800	10 800	11 000	11 100	10 700	10 700	8 300	8 300	8 500	13 300	12 600	12 200
5	10 500	10 000	9 800	11 200	10 700	10 700	9 000	10 000	10 100	9 300	9 500	9 400
6	11 300	12 300	12 500	10 900	10 500	10 700	10 400	9 700	9 500	7 500	9 400	9 600
7	10 400	10 700	10 700	11 900	11 000	10 800	9 200	10 000	10 000	8 400	8 800	8 800
8	12 100	10 200	10 200	12 100	13 000	13 100	8 500	9 300	9 400	7 300	9 000	9 400
9	7 600	9 900	10 200	10 500	10 900	10 800	14 400	9 300	8 400	10 900	10 600	10 700
10	10 500	12 000	12 100	14 200	11 000	10 500	7 200	10 400	10 900	7 800	8 700	8 700
11	14 200	10 500	9 700	7 500	8 500	8 600	8 300	9 400	9 300	7 000	9 400	9 800
12	10 400	12 000	12 200	6 500	7 500	7 800	11 000	9 300	9 000	7 900	9 100	9 400
13	12 600	9 800	9 500	8 300	8 400	8 200	7 200	9 800	10 200	11 000	10 800	10 900

TABLE 2.—STRESSES IN COLUMN NO. 1—(*Continued*).

Panel.	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Test No. 4.												
1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2	11 300	10 800	10 700	10 500	11 100	11 200	6 100	3 000	2 500	8 900	10 200	10 600
3	10 000	11 600	11 600	10 100	11 100	11 100	8 400	8 500	8 500	12 000	11 300	11 100
4	11 200	12 300	12 500	9 900	10 900	11 000	7 600	9 400	9 700	7 900	9 300	9 500
5	.....	.....	.....	12 800	11 100	10 700	.....	.....	.....	10 700	10 200	10 100
6	10 100	5 400	4 500	12 800	10 700	10 300	6 500	8 900	9 500	.....	.....	.....
7	8 000	10 600	11 000	10 200	9 700	9 700	.....	.....	.....	8 700	8 500	8 600
8	9 800	12 000	12 400	9 700	10 000	10 100	9 000	9 400	9 400	10 300	11 300	11 400
9	11 500	10 600	10 400	.....	.....	.....	8 500	10 700	10 900	7 900	11 100	11 500
10	8 600	8 300	8 100	6 800	8 700	9 100	10 800	7 500	7 000	9 300	10 700	11 000
11	.....	.....	.....	.....	.....	.....	9 500	9 000	8 900	13 000	12 700	12 600
12	10 900	10 400	10 300	10 700	9 900	9 600	.....	.....	.....	.....	.....	.....
13	11 000	10 700	10 700	11 100	10 600	10 500	7 300	9 800	9 600	12 700	11 900	11 800
Test No. 5.												
½	12 100	10 400	9 300	11 000	9 500	8 700	9 700	10 500	10 900	10 400	10 800	10 900
1	9 800	11 100	11 800	10 000	10 200	10 500	12 200	10 400	9 300	13 500	11 300	10 400
1½	7 900	8 500	8 600	5 800	8 200	8 900	7 300	8 400	8 900	7 400	7 900	8 300
2	11 800	11 400	11 100	10 900	10 600	10 600	8 500	8 600	8 700	9 800	10 200	10 400
2½	10 700	11 000	11 100	10 800	10 200	10 200	11 800	11 900	11 800	9 700	9 700	9 600
3	5 800	7 500	8 500	7 800	8 000	8 500	8 300	8 500	8 600	7 700	9 100	9 700
3½	8 000	8 100	8 500	7 400	8 100	8 500	7 800	8 600	9 000	7 500	7 600	7 700
4	15 400	13 000	12 400	13 500	13 000	12 800	11 600	11 800	11 800	9 900	9 900	9 900
4½	12 200	10 700	10 300	12 200	10 200	9 200	9 900	9 900	9 800	9 800	9 800	9 700
5	9 400	9 500	9 600	11 000	10 100	9 600	9 700	9 500	9 500	8 900	9 800	9 900
5½	9 900	10 700	11 100	10 500	10 900	11 000	10 900	9 100	8 300	8 700	8 800	8 900
6	7 400	7 300	7 200	8 100	9 200	9 700	13 800	11 900	11 100	11 100	10 600	10 200
6½	10 900	10 200	9 700	10 200	10 300	10 500	8 600	8 900	8 900	8 500	8 600	8 600
7	12 100	11 700	11 400	11 400	10 900	10 400	8 300	8 800	8 900	5 900	7 900	8 700
7½	9 900	9 200	8 900	9 600	9 400	9 500	6 500	7 600	8 300	4 600	6 400	6 900
8	9 800	8 600	8 300	8 100	9 700	10 400	11 400	9 500	8 500	12 700	10 800	10 100
8½	9 200	9 700	10 000	9 700	9 500	9 300	20 200	15 400	13 100	15 400	14 000	13 200
9	6 500	7 600	8 300	9 700	9 700	9 900	12 900	13 200	13 100	9 400	9 900	10 200
9½	10 000	10 400	10 600	12 900	9 500	8 100	4 600	7 800	8 900	4 600	7 300	8 500
10	16 400	12 900	11 200	17 000	14 400	12 900	8 000	8 600	8 700	6 800	8 000	8 600
10½	11 300	10 700	10 400	9 900	10 000	9 700	10 500	11 100	11 300	7 400	8 900	9 600
11	7 000	8 400	8 900	8 100	9 100	9 600	12 600	11 100	10 800	9 800	10 800	11 000
11½	9 700	8 900	8 600	11 900	10 500	9 700	8 200	11 100	11 700	8 900	9 700	9 800
12	14 000	12 400	11 600	11 900	11 000	10 400	6 900	7 500	7 800	8 100	8 800	9 400
12½	9 700	8 900	8 600	10 500	12 100	12 700	10 800	11 800	12 400	11 900	11 000	10 800

TABLE 3.—Stress in Wrought-Iron Bridge Posts.

Panel.	North Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	North Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. West side. Outer fiber.	Center of gravity.	Inner fiber.	South Channel. East side. Outer fiber.	Center of gravity.	Inner fiber.
Column No. 2, Test No. 6.												
1	4 600	6 500	7 300	6 400	7 900	8 500	4 300	8 300	9 700	5 000	6 800	7 400
2	7 300	7 300	7 300	8 300	8 400	8 400	9 800	9 000	8 800	9 000	8 000	7 600
3	5 500	5 900	6 000	7 100	7 800	8 000	7 700	8 800	9 200	6 700	7 600	7 900
4	6 200	6 100	6 200	6 100	6 900	7 100	8 400	8 800	9 000	8 200	8 000	8 000
5	7 200	6 500	6 100	8 800	8 600	8 500	7 500	8 800	9 300	8 700	7 800	7 500
6	6 800	6 900	6 900	8 400	8 400	8 400	7 100	7 600	7 800	6 900	7 200	7 400
7	7 500	7 900	8 000	7 100	7 400	7 500	7 500	8 200	8 500	7 500	7 400	7 300
8	6 400	7 400	7 800	6 200	7 200	7 600	7 700	7 900	8 000	6 600	6 400	6 100
Column No. 3, Test No. 7.												
1	9 300	9 000	9 000	8 400	9 500	10 000	10 000	9 000	10 100	8 200	8 000	7 900
2	10 200	10 000	9 900	10 200	11 500	11 900	8 400	9 100	10 000	9 100	8 700	8 500
3	8 800	8 700	8 600	9 800	10 000	10 000	10 000	9 700	9 600	9 400	9 600	9 600
4	10 500	9 100	8 600	8 500	9 500	9 900	8 600	9 800	10 200	9 300	7 500	6 900
5	10 200	10 400	10 500	8 800	9 600	9 900	10 100	9 800	9 700	10 400	10 200	10 200
6	10 600	10 000	9 800	10 300	10 000	10 000	10 400	10 400	10 400	10 000	9 900	9 900
7	10 300	9 900	9 700	9 600	9 900	10 000	8 700	9 500	9 700	10 900	10 000	9 700
8	9 800	9 600	9 500	10 200	10 900	11 000	7 600	9 600	10 300	7 800	9 900	10 600
Column No. 4, Test No. 8.												
1	9 000	9 800	10 000	10 000	10 700	11 000	8 200	10 000	10 800	9 000	9 200	9 300
2	8 700	9 300	9 500	10 100	10 800	11 100	10 100	11 600	12 100	8 600	9 400	9 700
3	9 300	9 800	9 900	10 800	11 200	11 400	10 000	10 200	10 200	7 400	8 400	8 700
4	9 500	9 700	9 800	11 100	10 200	10 000	9 700	11 100	11 600	7 100	8 200	8 500
5	10 200	10 500	10 600	11 900	12 400	12 600	8 400	9 400	9 800	7 700	8 800	9 200
6	11 600	12 000	12 100	12 400	13 600	13 900	9 500	10 800	11 100	7 400	9 500	10 200
7	12 800	12 500	12 400	13 400	14 200	14 600	7 100	8 300	8 700	6 400	6 700	6 700
8	8 600	11 600	12 700	13 000	11 300	10 700	6 600	8 500	9 200	6 200	7 800	8 400
Column No. 5, Test No. 9.												
1/8	14 600	11 700	10 800	12 600	11 700	11 400	7 400	8 100	8 400	7 400	7 900	8 000
1	11 300	12 000	12 300	11 100	10 000	9 600	6 700	8 500	9 200	5 100	6 800	7 500
1 1/8	11 200	11 100	11 100	11 100	10 800	10 700	5 700	6 200	6 300	6 600	6 900	7 100
2	13 100	12 600	12 400	12 900	12 400	12 230	8 000	8 000	8 000	7 800	8 200	8 300
2 1/8	12 700	12 000	11 700	13 100	11 500	11 100	9 300	9 200	9 200	8 700	8 400	8 300
3	11 800	12 000	12 200	11 400	12 200	12 400	8 200	8 400	8 400	6 300	8 000	8 600
3 1/8	13 500	10 500	9 300	11 600	10 700	10 400	8 200	8 200	8 200	8 400	8 300	8 200
4	14 300	13 100	12 700	14 300	12 300	11 500	7 300	7 400	7 500	7 400	8 700	9 200
4 1/8	11 600	11 600	11 500	12 300	12 300	12 300	9 000	9 000	9 000	8 700	8 700	7 800
5	12 600	12 400	12 300	11 700	10 400	10 000	8 300	8 400	8 400	7 400	8 500	8 100
5 1/8	13 700	12 100	11 500	13 500	12 100	11 400	8 300	8 300	8 300	8 400	8 400	7 600
6	12 600	12 700	12 700	13 500	12 500	12 200	7 600	7 600	7 600	7 200	7 000	7 100
6 1/8	13 100	11 900	11 500	11 400	11 400	11 500	8 400	8 400	8 400	10 500	9 000	8 500
7	11 100	11 300	11 500	11 200	12 300	12 600	7 900	8 000	8 000	6 500	8 100	8 600
7 1/8	10 600	10 100	9 900	11 000	10 300	9 900	8 000	8 200	8 200	5 900	7 900	8 700
8	10 300	12 300	12 800	11 300	12 300	12 400	8 500	8 600	8 600	10 800	11 800	12 200

TABLE 3—(*Continued*).

Panel.	North Channel. West side. Outer fiber.	North Channel. West side. Center of gravity.	North Channel. West side. Inner fiber.	North Channel. East side. Outer fiber.	North Channel. East side. Center of gravity.	North Channel. East side. Inner fiber.	South Channel. West side. Outer fiber.	South Channel. West side. Center of gravity.	South Channel. West side. Inner fiber.	South Channel. East side. Outer fiber.	South Channel. East side. Center of gravity.	South Channel. East side. Inner fiber.
Column No. 4*a*, Test No. 10.												
½	8 100	9 500	10 200	8 400	8 100	8 000	11 700	10 800	10 500	9 900	11 200	11 800
1	8 300	10 200	10 800	6 500	7 200	7 500	12 100	10 900	10 400	11 100	12 500	13 100
1½	8 400	9 400	9 700	7 400	8 200	8 400	12 100	11 400	11 100	11 000	11 400	11 600
2	8 700	8 800	9 000	6 200	7 200	7 500	10 800	10 500	9 700	9 300	11 700	12 600
2½	8 300	8 800	9 000	7 300	6 900	6 800	11 800	11 500	12 100	10 900	10 400	10 300
3	8 700	9 700	9 900	6 700	6 500	6 400	13 300	11 100	10 400	10 400	11 100	11 400
3½	8 300	8 000	8 000	7 300	7 300	7 400	13 600	11 600	11 100	13 000	12 400	12 400
4	8 200	9 700	10 100	6 800	6 900	6 800	14 300	13 500	12 700	13 100	11 100	10 500
4½	7 500	8 600	9 100	6 300	7 200	7 500	12 300	11 100	10 800	10 300	15 200	9 400
5	7 500	8 400	8 800	7 000	8 100	8 500	13 000	13 300	13 500	9 000	11 400	12 300
5½	6 300	7 200	7 500	5 900	6 100	6 200	12 700	12 300	12 100	12 300	10 700	10 100
6	6 800	7 800	8 200	5 300	6 800	7 200	14 200	14 200	14 200	11 000	11 200	11 300
6½	6 600	7 800	8 200	7 100	7 100	7 100	15 400	13 700	12 900	13 500	11 500	11 100
7	6 700	7 100	7 200	4 900	6 100	6 700	11 900	13 600	14 600	11 100	12 100	12 400
7½	6 100	7 600	8 300	4 800	8 100	9 400	12 300	12 400	12 400	9 300	16 200	11 100
8	6 000	8 200	9 000	8 400	6 800	6 200	10 700	12 200	12 800	10 000	16 300	11 100
Column No. 2*a*, Test No. 11.												
½	11 600	10 900	10 400	11 400	10 500	10 000	.....	.....	.....	.....	.....	.....
1	7 300	9 300	10 000	7 300	9 700	10 400	8 400	8 800	9 000	10 200	10 200	10 200
1½	11 600	9 900	9 600	11 600	10 900	10 000	8 500	10 000	10 600	11 400	12 000	12 200
2	8 900	8 700	8 600	10 500	10 300	10 000	10 200	9 000	8 600	11 500	11 200	11 000
2½	11 200	9 700	9 200	10 900	10 300	10 000	9 900	10 000	10 200	11 700	10 300	9 900
3	9 500	10 500	10 700	12 200	10 800	10 000	11 800	10 200	10 800	11 400	10 600	10 300
3½	10 500	11 000	11 000	12 100	10 600	10 000	10 400	8 600	8 100	9 700	9 600	9 400
4	9 800	9 500	9 400	10 600	9 900	9 600	9 800	9 400	9 200	10 400	10 600	10 500
4½	10 300	9 600	9 300	11 200	10 200	10 000	8 700	9 600	10 400	10 400	10 000	10 000
5	9 600	9 800	9 900	10 400	10 800	11 000	10 300	9 200	8 800	10 400	10 200	10 200
5½	12 200	11 600	11 200	12 000	11 100	10 700	12 500	10 400	10 000	12 200	11 900	11 700
6	10 100	9 700	9 700	9 900	9 700	9 700	9 100	10 400	10 900	10 300	10 700	10 700
6½	11 300	11 000	10 800	10 600	9 900	9 700	10 800	9 900	9 700	11 000	11 000	10 000
7	9 300	9 600	9 700	9 100	9 300	9 200	9 800	9 800	9 800	9 800	10 800	10 000
7½	10 200	10 900	11 400	9 100	9 400	9 500	9 300	9 600	9 800	9 900	10 700	10 800
8	9 900	9 800	9 700	11 100	10 000	9 600	8 800	10 300	10 900	10 800	10 700	10 800
Column No. 2*a*, Test No. 12.												
½	10 800	10 000	9 800	10 400	10 600	10 600	11 500	10 600	10 000	13 200	12 600	12 300
1	8 400	8 400	8 500	9 700	10 800	11 200	9 800	10 700	10 900	10 000	11 300	11 700
1½	8 200	8 000	7 900	9 200	10 600	10 900	9 200	9 600	9 700	8 800	10 000	10 200
2	9 200	9 800	10 000	9 900	10 900	11 300	11 400	10 000	9 400	12 200	11 400	11 000
2½	10 800	9 900	9 600	10 800	9 600	9 100	10 900	10 900	10 900	11 400	11 600	11 600
3	9 100	10 800	11 200	11 000	10 600	10 400	10 600	11 200	11 300	10 400	10 200	10 100
3½	10 700	10 900	10 800	12 000	10 600	10 000	10 600	10 900	10 900	10 200	10 600	10 600
4	10 000	8 800	8 400	11 400	9 400	9 400	8 900	10 000	10 300	8 800	10 300	10 900
4½	10 600	10 000	9 900	10 200	10 000	9 900	10 000	10 100	10 200	8 600	9 500	9 800
5	10 100	10 300	10 400	11 300	11 000	11 000	11 800	10 200	9 600	10 200	9 700	9 700
5½	10 900	11 300	11 300	11 400	10 600	10 300	13 000	11 000	10 400	11 000	9 500	9 000
6	11 500	10 800	10 500	9 500	9 000	9 000	9 500	9 900	10 000	9 700	9 500	9 500
6½	11 700	9 800	9 200	12 700	10 300	9 600	10 000	10 000	10 000	9 600	10 300	10 600
7	9 900	11 300	11 600	9 900	11 100	10 600	10 500	9 800	9 300	9 600	10 000	10 000
7½	8 000	9 700	10 300	15 400	15 400	15 400	7 800	10 000	10 600	6 800	8 500	9 000
8	10 000	9 800	9 900	11 700	10 100	9 900	8 800	10 400	11 100	9 400	9 400	9 400

TABLE 3—(*Continued*).

Panel.	North Channel.						South Channel.					
	West side.			East side.			West side.			East side.		
	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
	Column No. 2*a*, Test No. 13.											
1	4 500	7 000	8 600	7 000	8 600	9 100	9 800	10 800	11 100	10 300	12 300	13 000
2	7 600	9 000	9 500	10 300	10 400	10 600	10 900	10 100	9 700	12 800	12 600	12 600
3	7 000	8 600	9 100	9 400	10 100	10 300	11 300	11 300	11 300	11 600	12 600	13 000
4	8 100	9 100	9 500	10 400	10 800	10 900	9 300	10 400	10 700	9 600	11 600	12 500
5	8 200	9 500	10 000	10 000	10 200	10 200	10 900	8 900	8 100	10 600	10 700	10 800
6	10 000	10 400	10 600	10 500	10 000	10 000	8 500	9 600	10 000	9 600	10 400	10 800
7	9 800	9 800	9 900	10 200	10 300	10 300	9 500	8 800	8 700	9 900	10 000	10 100
8	8 000	7 800	7 700	12 300	10 300	9 200	7 700	8 700	9 100	8 700	10 000	10 500

Table 4 gives a number of the most marked deviations from average stress. The excess of the maximum fiber stress is given as a percentage of the average stress.

TABLE 4.—Maximum Observed Fiber Stresses in Flange Members of Columns.

Column Number.	Test Number.	Features of Lacing.	Method of Loading.	Percentage of excess of maximum fiber stress over average stress. Highest five values.
1	1	1¼ by 7/16 in., bolted.....	Central.	42, 41, 39, 31, 23
1	2	"	Slightly eccentric.	68, 64, 50, 49, 35
1	3	"	"	50, 41, 35, 32, 19
1	4	1 by ½ in., bolted.....	Central.	35, 29, 28, 27, 26
1	5	1 by ¼ in	"	67, 55, 49, 29, 27
2	6	.....	"	31, 23, 23, 21, 17
3	7	.....	"	20, 17, 12, 11, 9
4	8	.....	"	41, 29, 24, 22, 19
5	9	.....	"	43, 43, 38, 35, 35
4*a*	10	.....	"	53, 49, 42, 40, 37
2*a*	11	.....	"	21, 16, 13, 12, 11
2*a*	12	.....	Oblique arm, 1 in. (Fig. 5).	42, 42, 34, 33, 20
2*a*	13	.....	Oblique arm, 2 in. (Fig. 5).	35, 34, 22, 21, 21

In most cases the maximum stress was in the outer fiber of the channel; sometimes very high stresses were found in the inner fiber. Generally, the stress in the opposite channel was correspondingly less.

STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1

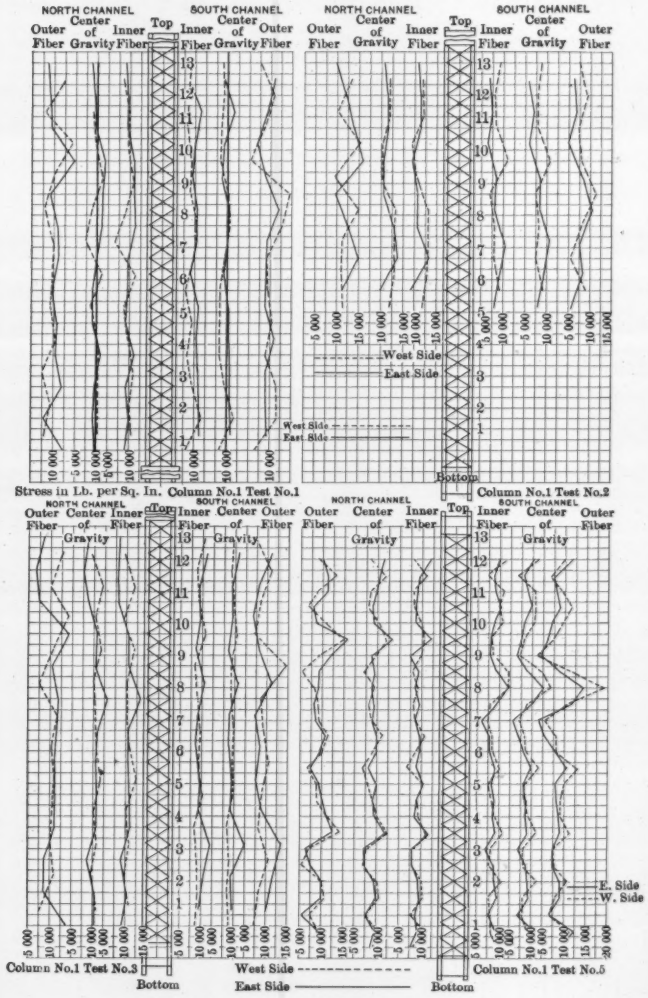


FIG. 6.

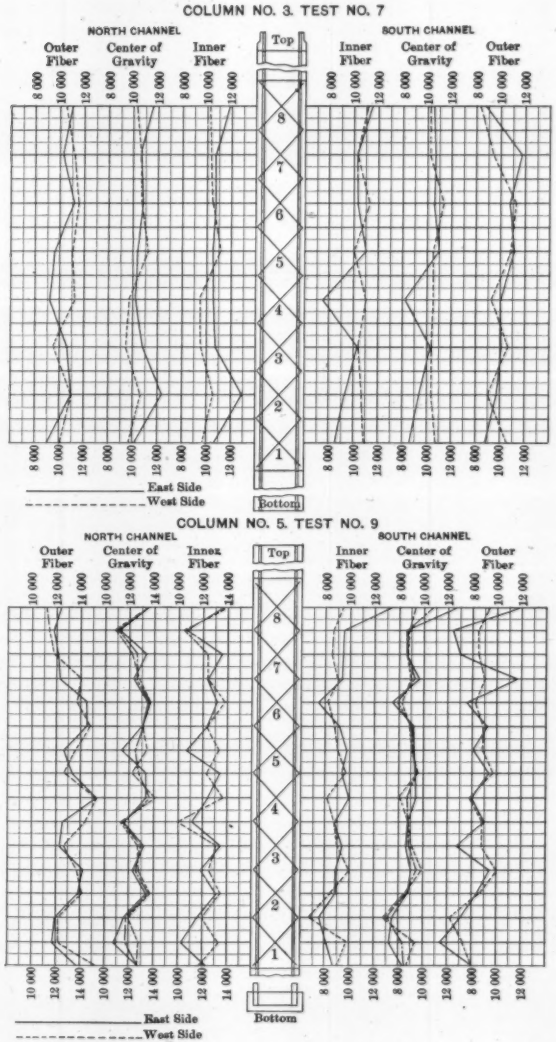

COLUMN NO. 3. TEST NO. 7
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
8 000
10 000
12 000
8
7
6
5
4
3
2
1
East Side
West Side
Bottom
COLUMN NO. 5. TEST NO. 9
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
10 000
12 000
14 000
8 000
East Side
West Side
Average
Bottom

FIG. 7.

Stress in Lattice Bars.—Table 5 gives the results of tests to determine the average stress in the various lattice bars of the columns. Tests 14 and 15 were tests on the lattice bars only. The distribution of stress over the cross-section of the bar is discussed in another place. The average stresses in the lattice bars are computed from the observed deformations, using a modulus of elasticity of 28 000 000 lb. per sq. in. for the steel column and 26 000 000 lb. per sq. in. for the wrought-iron columns. As might be expected, from the irregular variation of stress along the flange members of the columns, the stress in the lattice bars was found to vary greatly.

Table 6 gives the largest stresses observed and the corresponding transverse shear. The transverse shear given in this table is that which would cause a stress in the lattice bars equal to the maximum stress observed in any lattice bar, and was computed by doubling the transverse component of the maximum load observed on a lattice bar. In the case of obliquely loaded columns, the transverse component of the load was computed on the assumption that the load was applied

TABLE 5.—TOTAL STRESS, IN POUNDS, ON LATTICE BARS UNDER LOAD ON COLUMNS OF 10 000 LB. PER SQ. IN.

COLUMN NO. 1.

Lattice bar.	TEST NO. 5.		TEST NO. 14.		TEST NO. 15.	
	Front.	Back.	Front.	Back.	Front.	Back.
1	400*c*	400*c*	300*t*	400*t*	.....	.....
2	100*t*	600*t*	1 600*t*	400*c*	.....	.....
3	0	600*c*	800*c*	400*t*	.....	.....
4	100*t*	400*t*	300*t*	100*c*	.....	.....
5	100*c*	100*c*	.....	.....	.....	.....
6	200*t*	300*t*	.....	.....	.....	.....
7	100*c*	200*c*	.....	.....	.....	.....
8	200*t*	200*t*	.....	.....	.....	.....
9	0	300*c*	.....	.....	.....	.....
10	0	200*t*	.....	.....	.....	.....
11	100*c*	300*c*	.....	.....	.....	.....
12	0	100*t*	.....	.....	.....	.....
13	100*c*	200*c*	800*c*	0	.....	.....
14	100*t*	200*t*	900*t*	900*t*	2 500*t*	1 600*c*
15	0	300*c*	200*c*	1 000*t*	2 100*c*	.....
16	0	400*t*	800*t*	0	1 600*t*	200*c*
17	200*c*	200*c*	500*c*	300*t*	1 900*c*	.....
18	200*t*	100*t*	700*t*	1 000*t*	700*t*	1 100*t*
19	500*c*	200*c*	500*c*	200*t*	1 500*c*	.....
20	200*t*	300*t*	800*c*	200*t*	1 400*t*	700*t*
21	0	600*c*	800*c*	600*t*	1 500*c*	0
22	100*c*	2 000*t*	0	200*t*	700*t*	300*t*
23	200*t*	900*c*	300*c*	800*c*	1 300*c*	.....
24	200*c*	800*t*	200*t*	800*t*	600*t*	400*c*
25	200*t*	800*c*	0	0	2 700*c*	.....
26	200*c*	700*t*	400*t*	300*c*	400*t*	400*c*

TABLE 5—(*Continued*).

COLUMN NO. 2*a*, TEST NO. 11.

Lattice bar.	FRONT SIDE.		BACK SIDE.	
	Under.	Over.	Under.	Over.
1	700*c*	700*c*	1 300*c*	100*c*
2	100*c*	100*t*	200*t*	200*t*
3	3 000*t*	0	1 000*t*	0
4	100*c*	100*c*	800*c*	200*c*
5	3 000*t*	0	800*c*	300*c*
6	200*t*	200*c*	0	300*c*
7	3 700*t*	0	200*t*	200*c*
8	0	800*c*	800*c*	300*c*

COLUMN NO. 2*a*, TEST NO. 12.

1	1 500*c*	2 100*t*	200*c*	200*c*
2	200*c*	400*t*	2 300*t*	200*c*
3	800*t*	200*t*	2 750*t*	400*c*
4	700*c*	500*t*	2 750*t*	300*c*
5	1 600*t*	200*t*	2 750*t*	300*c*
6	0	0	2 800*t*	800*c*
7	1 400*t*	800*c*	4 100*t*	300*c*
8	100*t*	800*c*	300*t*	800*c*

COLUMN NO. 2*a*, TEST NO. 13.

1	1 000*c*	700*t*	800*t*	500*c*
2	50*c*	700*t*	2 650*t*	400*c*
3	700*c*	900*t*	2 750*t*	300*c*
4	500*c*	1 000*t*	2 100*t*	200*t*
5	800*c*	600*t*	3 150*t*	1 000*c*
6	1 000*c*	700*t*	4 100*t*	700*c*
7	700*c*	900*t*	1 500*t*	400*c*
8	1 600*c*	200*c*	2 000*t*	1 400*c*

through the center of the bearing blocks. This transverse component was then subtracted from the amount of shear which had been calculated from the deformation of the lattice bars as before noted, and the remainder has been tabulated under the heading "Transverse shear in column due to nominal central load."

Tests to Failure.—After the wrought-iron bridge posts had been tested for stress distribution under working loads, they were loaded to failure. Deformations were measured in the flange members of that part of the column on which the previous test had given the heaviest stress. Table 7 gives the result of the tests to failure. For all the tests of wrought-iron bridge posts, whether loaded centrally or eccentrically, the failures were very gradual. Final failure occurred near the middle or at the end. In the former case, high stresses in one channel had been shown by the deformation measurements at working

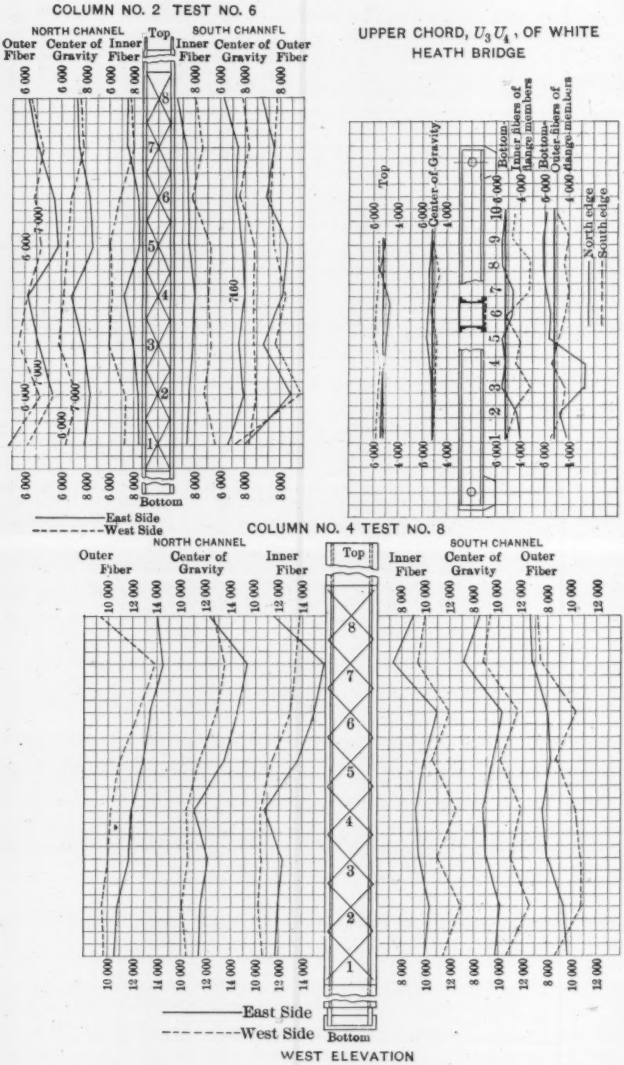

COLUMN NO. 2 TEST NO. 6
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Bottom
East Side
West Side
UPPER CHORD, U_3U_4, OF WHITE HEATH BRIDGE
Top
Center of Gravity
Bottom
Inner fibers of flange members
Outer fibers of flange members
North edge
South edge
COLUMN NO. 4 TEST NO. 8
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
East Side
West Side
Bottom
WEST ELEVATION

FIG. 8.

TABLE 6.—MAXIMUM OBSERVED AVERAGE STRESS IN LATTICE BARS OF COLUMNS NOS. 1 AND 2*a*.

Column number	1	1	1	2*a*	2*a*	2*a*
Test number	5	14	15	11	12	13
Method of loading	Central.	Central.	Oblique arm, 4 in. (Fig. 5).	Central.	Oblique arm, 1 in. (Fig. 5).	Oblique arm, 2 in. (Fig. 5).
Total load, in pounds	187 600	187 600	187 600	176 400	176 400	176 400
Lacing	Single, 63° 30′ with axis, 1 by ¼ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Single, 63° 30′ 1¼ by $\frac{7}{16}$ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.	Double, 45° 2½ by ⅜ in., riveted.
Maximum observed average stress in lattice bar, in pounds per square inch	8 200	3 000	4 900	3 900	4 350	4 350
Corresponding total stress on bar, in pounds	2 000	1 600	2 700	3 700	4 100	4 100
Corresponding transverse shear in column, in pounds	3 700	2 900	4 800	5 200	5 800	5 800
Shear due to known eccentricity of load, in pounds	0	0	3 000	0	1 000	2 000
Transverse shear in column due to nominal central load, in pounds	3 700	2 900	1 800	5 200	4 800	3 800
Ratio of shear to compression load	0.020	0.016	0.009	0.029	0.027	0.021
Next highest observed values of ratio of shear to compressive load	0.009 0.008 0.008 0.007	0.010 0.010 0.009 0.009	0.009 0.007 0.006 0.006	0.024 0.023 0.010 0.008	0.019 0.018 0.018 0.018	0.016 0.014 0.014 0.011

loads. In the latter case, a bending in one channel at working loads was noted by the instruments at the panel nearest the end of the column. In two of the three columns in which failure took place in the end of the column, as the instruments did not show over-stress in the laced portion of the column, the injured ends were removed, new end connections were put on, and the columns were retested as Nos. 2*a* and 4*a*.

TABLE 7.—RESULTS OF TESTS TO FAILURE.

Column Number..	2	3	4	5	2*a*	4*a*
Test number.....	6	7	8	9	13	10
Method of loading	Central.	Central.	Central.	Central.	Oblique arm. 2 in. (Fig. 5).	Central.
Load at failure, in pounds.....	466 000	480 000	452 000	492 000	475 000	475 000
Average stress at failure, in pounds per square inch..	26 400	27 200	25 700	27 900	26 900	26 900
Average stress at failure in tests of short pieces of flange members in pounds per inch	36 800	36 800	36 800	36 800	36 800	36 800
Percentage of excess of ultimate strength of short pieces over column strength.	39.5	35.5	43.5	32.0	37.0	37.0
Load at first sign of yielding, in pounds.....	317 000	264 000	256 000	264 000	.....	.....
Average stress at first sign of yielding, in pounds per square inch.	18 000	15 000	14 500	15 000	.....	.....
Method of failure, and remarks....	End failed.	End failed.	End failed.	Bowed at middle in plane perpendicular to lacing.	Buckled near bottom in plane parallel to lacing.	Bowed at middle in plane perpendicular to lacing.

Failure by Buckling of Lattice Bars.—In one of the tests of Column No. 1, the lattice bars failed by buckling suddenly and without warning. As tested, the column was fitted with the light lattice bars (1 by ¼ in.) riveted in place. The test had in view the trial for stress distribution under a slight obliquity, which was not carefully determined. No measuring instruments were in place. A preliminary load was being applied. When the load reached 150 000 lb. (8 060 lb. per sq. in. of cross-section), the alternate lattice bars in the upper half of the column buckled. A failure of this kind was quite unexpected at such a low load. Although an observer was watching the column, the failure was so sudden that he was unable to follow the movement of the parts. In this respect it was quite in contrast to the

failure of the other columns. The machine was at once stopped. Little damage was done to the column, except to the lacing bars. The webs were easily straightened, new lacing bars put on, and the column was used in another test.

Tests to destruction under compression had previously been made on lattice bars like those used in this column, and the results, in the absence of other data, may be useful in estimating the load carried by the lattice bars at failure. Under conditions of loading similar to the conditions found in column lattice bars, these sample bars failed under an average load of 2 100 lb. Assuming that the bar in this column which first failed was carrying 2 100 lb. when failure occurred, the transverse shear in the column may be computed. The following is a tabulated statement of the results of this test:

Column.	Compressive load, in pounds.	Lacing.	Manner of loading.	Probable maximum load on lattice bar, in pounds.	Corresponding shear in column, in pounds.	Ratio of transverse shear to compression load.
No. 1.....	150 000	Single 63° 30′ 1 by ¼, riveted.	Very slight obliquity.	2 100	3 760	0.0251

The column, now riveted up with the heavier lacing bars (1¼ by $\frac{7}{16}$ in.), was loaded obliquely, as shown in Fig. 5, with 300 000 lb. (15 900 lb. per sq. in. of cross-section), and the stresses in the lacing bars were measured. No sign of failure was apparent, and, from the data given in a succeeding paragraph, it appears that under this load the lattice bars must have been stressed to about three-quarters of their ultimate strength in compression.

Cross-Bending Tests of Columns.—Cross-bending tests were made on one of the wrought-iron columns and on Column No. 1. The tests were made in an Olsen 200 000-lb. testing machine fitted for testing beams 20 ft. long. The columns were supported at the ends and loaded at the center with a light transverse load. The column was placed first with the plane of the lacing perpendicular to the load, and then with the plane of the latticing parallel to the load. The lattice bars used in the tests of Column No. 1 were 1¼ by $\frac{7}{16}$ in. in cross-section; in one test they were bolted in place and in another they were riveted. The deflection at various points along the beam was measured

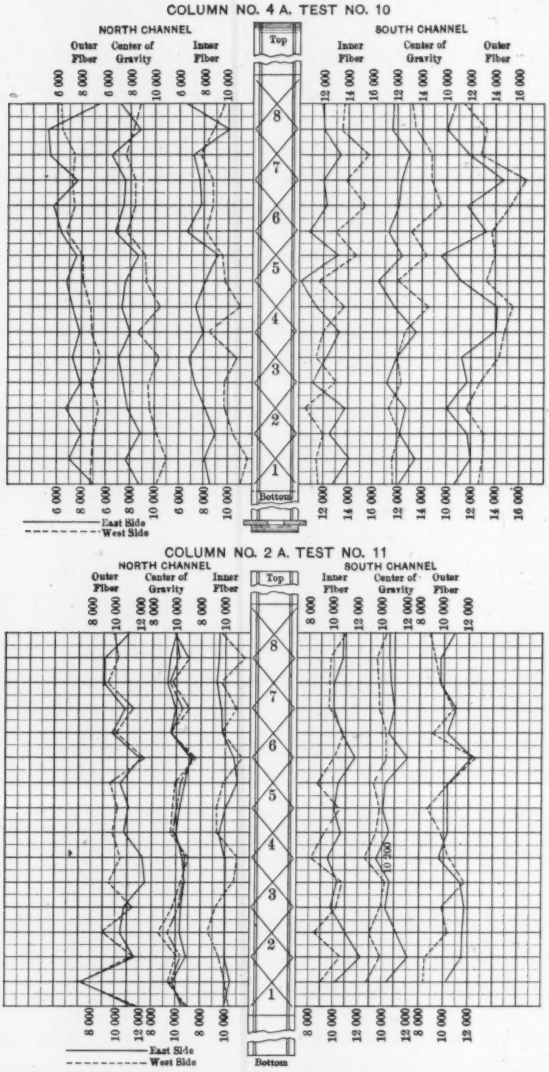

COLUMN NO. 4 A. TEST NO. 10
NORTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
SOUTH CHANNEL
Inner Fiber
Center of Gravity
Outer Fiber
Top
Bottom
East Side
West Side
COLUMN NO. 2 A. TEST NO. 11
NORTH CHANNEL
SOUTH CHANNEL
Top
Bottom
East Side
West Side

FIG. 9.

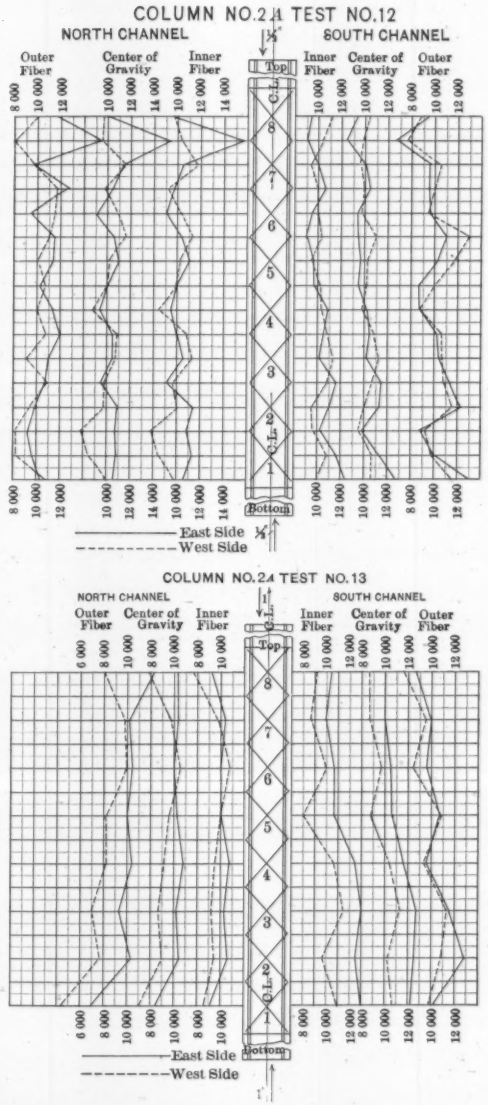

COLUMN NO. 24 TEST NO. 12
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
Bottom
East Side
West Side
COLUMN NO. 24 TEST NO. 13
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Center of Gravity
Inner Fiber
Top
Bottom
East Side
West Side

FIG. 10.

with Ames test gauges, and the actual curve assumed by the column under transverse load was thus determined. The theoretical elastic curve was computed from the common theory of flexure, not counting the lattice bars in the calculation of the moment of inertia. Fig. 2, Plate XII, shows the deflectometers and extensometers on Column No. 1 under the cross-bending test. Fig. 14 shows the deflection curves given by the column under transverse load and also the computed elastic curves.

It will be noted that when tested with the lacing vertical, Column No. 1 shows much greater deflection than that computed from the usual beam formula, while the stiffer wrought-iron column shows a much closer agreement with the curve, the heavy lacing apparently adding stiffness.

Field Tests of Columns.

Description of Bridge.—The bridge on which the field tests were made is an eight-panel single-track Pratt truss which spans the Sangamon River near White Heath, Illinois. The bridge is on the line of the Illinois Central Railroad between Champaign and Clinton, Illinois. The span measured 158 ft. 6 in. Fig. 11 gives a diagram of the bridge and Fig. 3, Plate XII, is from a photograph of the bridge.

Loads.—The test load applied to the bridge consisted of a mogul locomotive and tender followed by a loaded coal car and a caboose. Fig. 11 shows the test train, with dimensions and weights. This train was furnished through the courtesy of the railroad.

Members Investigated.—The members studied for stress distribution were Posts U_2L_2 South, U_3L_3 South, U_3L_3 North, and upper chord U_3U_4 South. The bridge diagram, Fig. 11, shows the location of the members. The posts were made up of two steel channels, double laced, while the top chord was made up of two built-up channels with a cover-plate on top and lacing across the bottom. The cross-section of the members tested is shown in Fig. 1. Fig. 12 shows the post and the chord.

Measurement of Deformation.—Ames test gauges were used as extensometers, and the method of attachment was the same as in the laboratory tests of columns. The method of reduction of instrument readings to stresses at the extreme fibers of members was also the same.

Routine of Tests.—As in the laboratory tests, the stress distribution in the channels and the lattice bars was studied. The method of

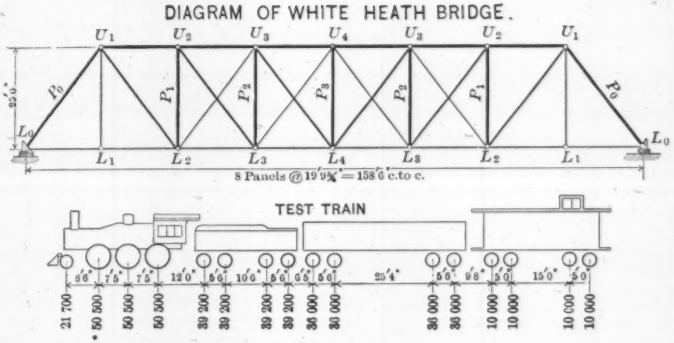


DIAGRAM OF WHITE HEATH BRIDGE.
U_1 U_2 U_3 U_4 U_3 U_2 U_1
25'0"
P_0 P_1 P_2 P_3 P_2 P_1 P_0
L_0 L_1 L_2 L_3 L_4 L_3 L_2 L_1 L_0
8 Panels @ 19'9½" = 158'6" c. to c.
TEST TRAIN
8'6" 7'5" 7'5" 12'0" 5'6" 10'6" 5'6" 5'6" 5'6" 25'4" 5'6" 9'8" 5'0" 15'0" 5'0"
21 700 50 500 50 500 50 500 39 200 39 200 39 200 39 200 36 000 36 000 36 000 36 000 10 000 10 000 10 000 10 000

FIG. 11.

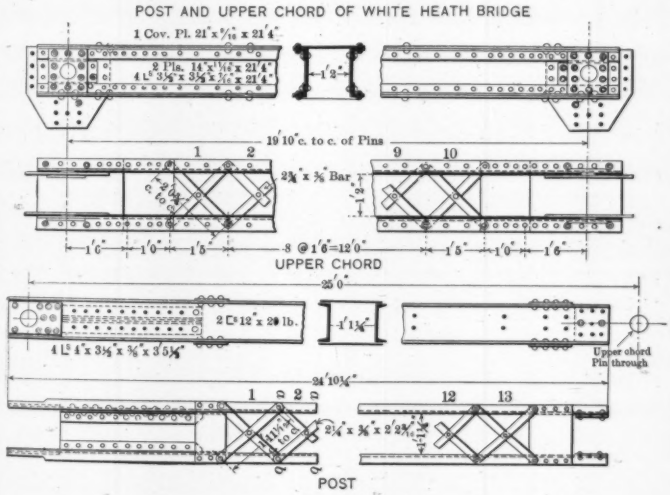

POST AND UPPER CHORD OF WHITE HEATH BRIDGE
1 Cov. Pl. 21" x 5/16 x 21'4"
2 Pls. 14" x 1/16 x 21'4"
4 L 3½" x 3½" x 5/16 x 21'4"
1'2"
19'10" c. to c. of Pins
1 2 9 10
2¾" x ⅝" Bar
1'2"
1'0" 1'0" 1'5" 8 @ 1'6" = 12'0" 1'5" 1'0" 1'6"
UPPER CHORD
25'0"
2 C 12" x 20 lb.
1'11½"
4 L 4" x 3½" x ⅝" x 3'5¼"
Upper chord Pin through
24'10¼"
1 2 12 13
2¼" x ⅝" x 2'2½"
POST

FIG. 12.

procedure was as follows. Instruments were placed on some portion of the column to measure the deformation over a short gauge length, and a reading was taken. The test train was then run upon the bridge to a given position (one approximating the maximum load on the member under test), and the instruments were read again. The train was then run off the bridge, and the instruments were again read. This procedure was repeated several times, at least three applications of the load being made and frequently several more. The instruments were then moved to another part of the column, and that part was tested. Observations were made on both flange members and lattice bars. The tests covered a period of eight days.

TABLE 8.—STRESSES IN POSTS OF WHITE HEATH BRIDGE.

	North Channel.						South Channel.					
	East side.			West side.			East side.			West side.		
Panel.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.	Outer fiber.	Center of gravity.	Inner fiber.
Post U_3L_3 South.												
1*	5 000	5 300	5 400	2 600	2 700	2 700	4 900	5 000	5 000	2 300	2 500	2 600
2	3 400	3 500	3 500	1 900	1 600	1 500	3 300	3 400	3 400	1 600	1 800	1 800
3	3 500	3 800	4 000	1 800	1 900	2 000	2 600	2 900	3 100	2 600	2 200	2 100
4	3 700	3 400	3 300	2 800	3 000	3 000	1 600	3 100	3 600	2 400	2 800	2 960
5	3 500	3 900	4 000	2 900	2 900	2 900	2 800	3 000	3 100	2 200	2 600	2 800
6	3 600	3 600	3 600	3 400	3 800	3 900	3 400	3 400	3 400	4 100	3 400	3 200
7	2 600	3 200	3 400	3 300	3 100	3 000	2 400	3 000	3 300	2 600	2 700	2 800
8	3 300	3 200	3 200	3 900	3 600	3 500	3 600	3 400	3 300	3 700	3 900	3 900
9	3 200	3 100	3 000	3 600	3 300	3 200	1 700	2 500	2 800	2 900	3 300	3 400
10	2 800	2 700	2 700	3 100	3 300	3 300	2 700	2 400	2 300	3 300	3 700	3 900
11	3 300	2 900	2 800	4 500	3 400	3 100	2 900	2 000	1 700	3 500	3 900	4 100
12	1 400	2 000	2 300	3 100	3 200	3 200	1 800	1 800	1 800	4 800	4 700	4 600
13	1 300	1 500	1 500	2 800	2 700	2 600	2 600	2 900	3 100	5 200	4 900	4 800
Post U_3L_3 North.												
1	4 900	4 900	4 900	2 600	2 500	2 500	4 800	4 900	4 900	2 100	2 700	2 800
1½	4 800	4 700	4 700	2 900	2 800	2 700	3 100	4 900	5 700	900	1 500	1 700
2	4 800	4 400	4 300	2 600	2 500	2 500	4 900	4 700	4 600	2 700	1 700	1 300
2½	4 000	3 900	3 900	2 700	2 800	2 900	4 300	3 700	3 500	2 300	1 800	1 600
3	3 900	4 200	4 300	2 800	2 900	2 900	.....	.....	.....	.....	.....	.....
3½	3 900	3 900	4 000	2 400	2 500	2 500	3 200	3 800	4 100	2 400	2 500	2 500
4	3 400	4 600	5 000	2 700	2 700	2 700	3 900	3 700	3 700	3 500	3 200	3 100
4½	3 800	3 400	3 200	2 500	2 300	2 200	3 500	3 500	3 400	3 100	3 000	3 000
5	3 900	3 700	3 600	3 700	3 700	3 700	3 100	3 600	3 800	1 400	2 100	2 400
5½	4 100	3 600	3 400	4 000	3 400	3 200	3 500	3 900	4 100	3 500	2 900	2 800
6	3 600	3 200	3 100	3 400	3 300	3 200	3 700	3 500	3 400	3 900	3 600	3 500
6½	3 300	3 300	3 300	3 600	3 500	3 500	4 500	4 700	4 800	2 000	2 400	2 600

* No explanation for the high values in Panel 1 has been found. Five determinations of stress were made, including the removal and re-attachment of instruments.

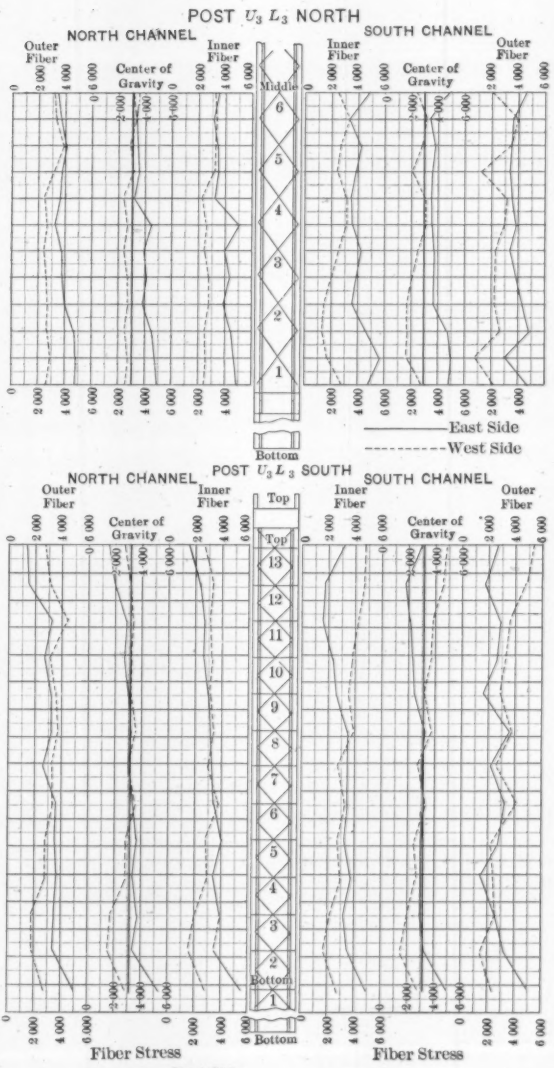

POST U_3 L_3 NORTH
NORTH CHANNEL
SOUTH CHANNEL
Outer Fiber
Inner Fiber
Center of Gravity
Middle
Bottom
East Side
West Side
POST U_3 L_3 SOUTH
NORTH CHANNEL
SOUTH CHANNEL
Top
Bottom
Fiber Stress
East Side
West Side

FIG. 13.

Results of Tests for Stress Distribution in Channels.—Table 8 gives the results of the tests to determine the stress distribution and variation in the channels of the bridge posts, and Table 9 gives those for the top chord. The stresses given are calculated from the observed deformations, using a modulus of elasticity of 30 000 000 lb. per sq. in. The conditions of measurement of deformation were much the same as in the laboratory tests. The stress noted is the average stress over a space of 4½ in. on either side of the point indicated.

TABLE 9.—Stresses in Upper Chord of White Heath Bridge.

Lower Side (Laced).				Upper Side (Cover-plate).				
North Channel.		South Channel.		Distance from end, in inches.	North edge.	Over north web plate.	Over south web plate.	South edge.
Panel.	Outer fiber.	Inner fiber.	Outer fiber.	Inner fiber.				
1	4 300	3 900	5 000	5 500	33	5 500	6 000	5 200	5 700
2	4 900	4 300	4 500	4 800	57	.....	.....	5 200	5 500
3	2 800	5 500	3 400	4 500	81	5 200	5 400	5 900	5 300
4	2 800	5 100	4 200	5 300	105	5 000	6 100	5 700	6 000
5	5 700	5 400	3 900	4 800	129	4 800	5 900	5 900	5 900
6	5 500	4 800	5 100	4 200	153	5 400	5 700	5 500	5 600
7	5 900	4 700	3 400	3 600	177	5 000	5 000	5 700	5 600
8	6 200	5 300	3 300	4 400	.....	.....	.....	.....	.....
9	6 000	5 300	4 700	3 800	.....	.....	.....	.....	.....
10	5 900	5 100	4 400	4 900	.....	.....	.....	.....	.....

TABLE 10.—Maximum Observed Fiber Stress in Flange Members of Columns in White Heath Bridge.

Column number	U_3L_2 South F 1	U_3L_2 North F 2	U_3U_3 South F 3
Test number			
Lacing	Double, 45°, riveted at crossing.	Double, 45°, riveted at crossing	Cover-plate on top. Double, 45° on bottom.
Maximum observed compressive stress in an extreme fiber, in pounds per square inch	5 200	5 700	6 200
Percentage of excess maximum observed stress over average, highest five values.	73*	64	20
	63	48	19
	60	41	17
	57	31	17
	47	31	17

* No values from Panel 1 have been included in this table, as no explanation is known of the high stresses indicated in that panel.

Figs. 8 and 13 show graphically the stress distribution and variation. In these figures the full lines give the stresses at the west side (front), and the dotted lines the stresses at the east side (back).

In Table 10 are given a number of the highest observed fiber stresses. The excess of the maximum fiber stress is given as a per-

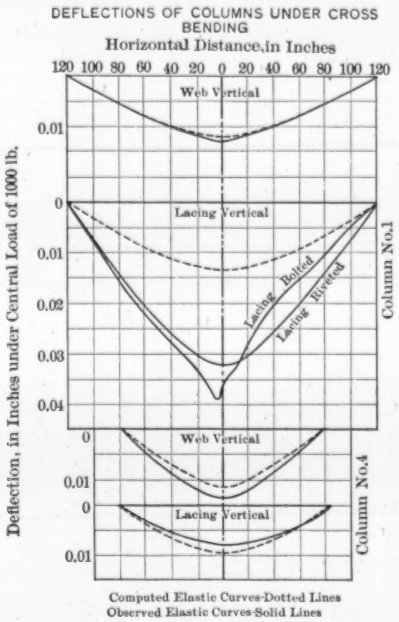


FIG. 14.

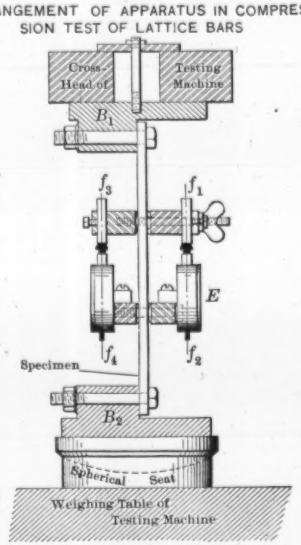


FIG. 15.

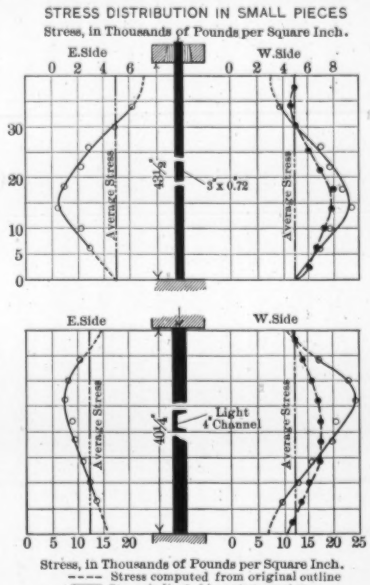


FIG. 16.

centage of the average stress. At most sections the maximum stress was in the outer fiber of the channel, but in some cases it was found at the inner fiber.

In the tests of the bridge posts an attempt was made to determine the stresses in a few of the lattice bars. These stresses were very small, and the precision of the extensometer was not sufficient to measure them with any great degree of accuracy. It should be noted that the lacing of the posts in this bridge was double, and the bars were riveted together at their intersection. In several cases it was found that a lattice bar under load bent in the shape of a very flat S curve, the point of attachment to another lacing bar, at the middle, being a point of inflection.

Special Tests on Bridge Columns.—Tests were made on the batten plates at the top of one of the posts, and under load a slight bending of the plates between channels was found. The bending took place in a horizontal plane.

In one post the change of stress was observed as the locomotive and train moved across the bridge. Extensometers were placed at *aa, bb* (Fig. 12), and on the floor beams, and the changes in reading were noted as the train moved across the bridge. In the inner channel of the post, tension was set up as the locomotive came opposite the post.

TESTS OF LATTICE BARS, SMALL COLUMNS, AND COLUMN MATERIAL.

Compression Tests of Lattice Bars.—Many of the lattice bars in a column, as they transmit stress from one flange member of the column to the other, are under compression. To study the action of lattice bars under compression, a series of tests on single lattice bars was made. Fig. 15 shows the arrangement of the apparatus. The lattice bar was tightly bolted to the blocks, B_1 and B_2. The upper block, B_1, was fastened to the cross-head of a testing machine, and the block, B_2, was pressed against the weighing table of the testing machine. A spherical-seated bearing block was used, to insure an even bearing. Ames test gauges, E, mounted on suitable frames, were attached to the lattice bar over a short gauged length. From the readings of these gauges, the deformation of the extreme fiber of the bar was computed.

In this test of lattice bars, the load was applied with an eccentricity approaching that to be expected in a column for the lattice bars which

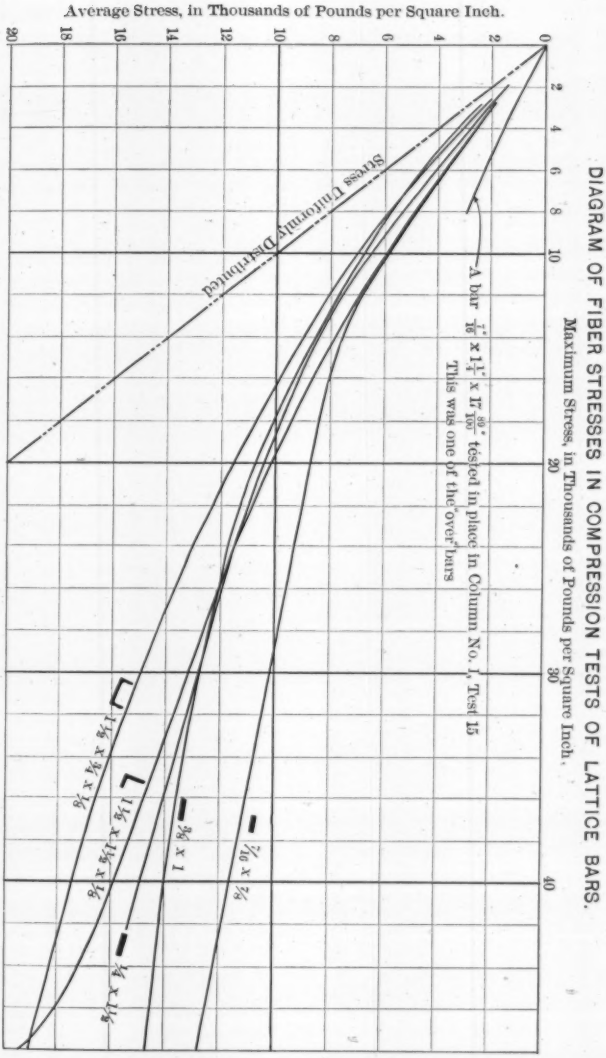


DIAGRAM OF FIBER STRESSES IN COMPRESSION TESTS OF LATTICE BARS.
Maximum Stress, in Thousands of Pounds per Square Inch.
Average Stress, in Thousands of Pounds per Square Inch.
Stress Uniformly Distributed
A bar 7/16" x 1 1/4" x 17 89/100" tested in place in Column No. 1, Test 15
This was one of the "over" bars
1 1/8 x 3/4 x 1/8
1 1/8 x 1 1/8 x 1/8
3/8 x 1
7/16 x 7/8
1/4 x 1 1/8

FIG. 17.

are next to the flange member (here designated "under" bars). The lattice bars outside of these "under" bars are here designated "over" bars. The stress distribution across the section of the "over" bars, which are under compression, is probably more uneven than the stress distribution found in these tests. However, these tests give some idea of the relative behavior of lattice bars of various proportions, and of the large eccentricity of loading of all lattice bars.

Lattice bars of the following cross-sections were tested: Flat bars 1½ by ¼ in., 1 by ⅜ in., ⅞ by $\frac{7}{16}$ in.; angles 1½ by 1½ by ⅛ in.; channels 1½ by ¾ by ⅛ in. Several channel and angle lattice bars were tested with ends flattened and ribs turned inward, to minimize the eccentricity of loading. Bars of the following lengths between centers of rivet holes were tested: 8½ in., 13½ in., and 20 in. The rivet holes were ½ in. in diameter. All bars were tested in a Philadelphia Machine Tool Company's 100 000-lb. testing machine, and loads and extensometer readings were taken to failure.

Observations were also made on the behavior of a lattice bar in a column under load, with a view to determine the distribution of stress over the section. Column No. 1 was loaded obliquely. The instruments were placed on an "over" bar which had been found to carry a high compressive stress, and readings were taken to determine the distribution of stress across the section.

When Column No. 1 was under cross-bending test, observations were made to determine the stresses transmitted by lattice bars and their distribution over the section of the bars. Extensometers were placed successively on most bars under compression on one-half of the column, and on some bars which were under tension. In both of these tests the bars were 1¼ by $\frac{7}{16}$ in., and were riveted.

Results of Tests of Lattice Bars.—The results of the tests of single lattice bars are given in Figs. 17 and 18, and in Tables 11 and 12. Fig. 17 shows the ratio of maximum to average stress in the bars 13½ in. long between centers of rivet holes. It also gives the result of the test of stress distribution in a lattice bar of Column No. 1. Table 11 gives the stresses at failure of the various bars tested singly. The average stress on the various bars, which corresponds to a maximum fiber stress of 40 000 lb. per sq. in., as taken from these tests, has been noted and is given in Table 12. The results of the tests to failure are shown graphically in Fig. 18. The angle and channel bars

tested with flattened ends and ribs turned inward failed in the flattened part at loads no greater than the bars fastened with ribs turned outward.

TABLE 11.—COMPRESSION TESTS OF LATTICE BARS.

Average of two specimens.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	$\frac{l}{r}$	Average stress at failure, in pounds per square inch.	Stress at failure for steel of 40 000 lb. per sq. in. yield point, in pounds per square inch.*
1½ by ½, flat	20	277	9 900	8 900
1 by ⅜, flat	20	184	12 900	12 200
⅞ by 7/16, flat	20	158	14 500	15 000
1½ by ¾ by ⅜, channel	20	90	20 800	19 400
1½ by 1½ by ⅜, angle	20	43	22 600	20 100
1½ by ½, flat	13½	187	15 400	13 800
⅞ by 7/16, flat	13½	107	16 300	16 800
1 by ⅜, flat	13½	124	16 900	15 900
1½ by ¾ by ½, channel	13½	61	Bolt sheared.	.....
1½ by 1½ by ½, angle	13½	29	Bolt sheared.	.....
1½ by ½, flat	8½	118	17 300	15 500
⅞ by 7/16, flat	8½	67	18 100	18 700
1 by ⅜, flat	8½	78	18 300	17 200
1½ by ¾ by ½, channel	8½	38	Bolt sheared.	.....
1½ by 1½ by ½, angle	8½	18	Bolt sheared.	.....

*The values in this column were obtained by multiplying the observed stress at failure by $\frac{40\,000}{\text{yield point determined from tests}}$.

TABLE 12.—AVERAGE STRESS IN LATTICE BARS WHICH CORRESPONDS TO A MAXIMUM FIBER STRESS OF 40 000 LB. PER SQ. IN.

Section of bar, in inches.	Distance from center to center of rivet holes, in inches.	Corresponding average stress, in pounds per square inch.
⅞ by 7/16, flat	13½	11 600
1 by ⅜, flat	13½	14 000
1½ by ½, flat	13½	14 900
1½ by 1½ by ½, angle	13½	15 900
1½ by ¾ by ½, channel	13½	17 500

Table 13 gives the results of the test for stress distribution in the lattice bars of Column No. 1 as it was stressed in cross-bending.

Tests of Small Columns.—Tests of two small compression pieces were made in order to study the effect of slight bends and kinks in the column upon the distribution of stress. The deviation from a straight line, in these nominally straight pieces, was measured before the load was applied. The deformations on two opposite faces for a given load were measured. The extensometer was similar to that used on the

single lattice bar tests. The instrument was shifted from one position to another along the column. The columns were finally loaded to failure. One of the columns was a flat piece of steel, 3 by 0.72 in. in cross-section, and 40 in. long. It was held at the upper end by wedge grips in the cross-head of the machine and at its lower end rested on a spherical-seated block. The second compression piece was a 4-in. channel 40 in. long. The ends were planed square; the upper end bore on a flat compression plate in the iron head of the machine, and the lower end rested on a spherical-seated block.

TABLE 13.—STRESS IN LATTICE BARS OF COLUMN NO. 1 UNDER CROSS BENDING.

Column tested as a beam centrally loaded over span of 19 ft. 8 in.; lattice bars $1\frac{1}{4}$ by $\frac{7}{16}$ in.; 17.89 in. center to center of rivet holes; rivets $\frac{1}{2}$ in. in diameter.

Bar.	Maximum fiber stress, from Ames dials.	Average stress, from Ewing extensometer.	Ratio, maximum to average.
	"OVER" BARS IN COMPRESSION.		
14*E*	6 000	2 600	2.31
16*E*	8 300	2 000	4.15
18*E*	4 300	1 800	2.39
20*E*	7 700	1 400	5.50
22*E*	7 000	2 900	2.42
24*E*	8 900	2 200	4.04
			Average, 3.47
	"UNDER" BARS.		
15*W*	4 300*c*	2 500*c*	1.72
25*E*	3 800*t*	3 100*t*	1.23
17*E*	3 900*t*	3 000*t*	1.30
25*W*	3 900*c*	2 800*c*	1.39
19*W*	4 500*c*	2 900*c*	1.55
			Average, 1.44
	"OVER" BARS IN TENSION.		
16*W*	7 000	3 000	2.33
18*W*	7 000	2 200	3.19
20*W*	4 500	2 400	1.87
22*W*	5 700	2 200	2.59
24*W*	7 000	2 700	2.59
26*W*	3 800	2 200	1.73
			Average, 2.38

Fig. 16 gives the results of tests of the small columns. The dotted line represents the maximum fiber stress computed by considering the eccentricity of loading at any cross-section to be equal to the deviation of that section from a straight line connecting the ends of the column. The deflections were slight, and were neglected in the calculations. The solid line represents the stresses on the two sides of the column, as determined from the extensometer readings.

TABLE 14.—TENSION TESTS OF MATERIAL FROM COLUMNS.

Test piece from:	Material.	Number of test pieces.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Average Elongation. Percentage.
Column No. 1					
Angles	Steel.	2	43 300	61 600	37
$\frac{7}{16}$ by 1½-in. lacing bar	Steel.	2	40 700	58 100	42
1 by ½-in. lacing bar	Steel.	1	42 400	62 000	38
Channels of wrought-iron columns	Wrought iron.	3	30 700	46 800	17

Tests of Column Material.—Table 14 gives the results of the tensile tests of samples of material from various parts of the flange members of the columns, and also of tension tests of lattice bars. Table 15 gives tension tests of lattice bars like those used in the compression tests of lattice bars.

TABLE 15.—TENSION TESTS OF LATTICE BARS.

Shape.	Dimensions of cross-section of whole bar, in inches.	Number of specimens tested.	Average stress at yield point, in pounds per square inch.	Average stress at ultimate, in pounds per square inch.	Percentage of elongation in 2 in.
Channel.	1½ by ¾ by ½	1	43 000	57 600	36.5
Angle.	1½ by 1½ by ⅛	2	45 000	59 300	24.5
Flat.	$\frac{7}{8}$ by $\frac{7}{16}$	2	38 700	57 000	45.5
Flat.	1 by ⅜	2	42 400	61 700	45.8
Flat.	1½ by ¼	2	44 600	60 800	42.8

COMPARISON AND DISCUSSION.

The Action of Built-up Compression Pieces.—In analytical discussions of column action, the stress is usually assumed to vary uniformly from a minimum on one side of the cross-section to a maximum on the opposite side, and the whole cross-section of the column is considered to act as a unit. The longitudinal axis of the column is also considered to take a definite elastic curve under load. In the

derivation of most column formulas, it is assumed that the amount of deflection of the elastic curve from the original position of the axis is an important element in fixing the maximum stress in the column. Although these assumptions are generally used as the basis of column formulas, it may be well to consider whether conditions may not exist, in columns of ordinary form and dimensions, which will render doubtful the general applicability of some of these assumptions, and will dwarf the effect of others. At any rate, it seems worth while to consider the effect of other conditions in a built-up member. It must be borne in mind that the built-up column is subject to imperfections of fabrication, and that some crookedness and eccentricity must exist.

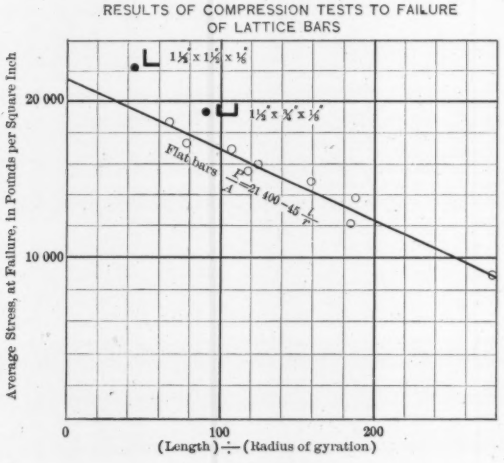


FIG. 18.

The component parts of the column may be relatively slender and flimsy. Whether there is integrity of cross-section under load, is a question. In the tests herein described, the amount of deflection from the original axis, for loads up to a point somewhat below incipient failure, was found to be slight (generally between 0.04 and 0.1 in.), much smaller than necessary to account for the stresses observed in the columns.

The action of short columns at failure may be expected to be different from that of longer columns, although the stresses up to incipient failure may be the same. Granting that the conditions of

non-straightness are such that the distribution of stress over the cross-section is the same for the two lengths of column, and that the deflection of the column is so slight as not to affect materially the stresses developed, the longer column will be in more danger of immediate and sudden collapse after the yield point of the material in any fiber has been reached, and the total load carried before complete failure will, in general, be less. This is because, in a ductile material, after the stress at one side of the column has passed the yield point, the total resistance of the section to compression will increase, while the resistance to cross-bending may not. Under the conditions named, the bending moment due to eccentricity will be the same until the yield point in some fiber is reached. After yielding begins, the greater deflection in the longer column rapidly increases its relative eccentricity, and more rapid failure may be expected than with the shorter column.

Indications of Data.—It will aid in the interpretation of the data of the distribution of stress over the channel members of the columns to point out a few simple indications. Reference may be made to the diagrams in Figs. 6, 7, 8, and 13, and to Tables 2, 3, 8, and 9.

1.—Any lack of agreement between the average load stress and the average of the stresses given for the four centers of gravity of channel flanges may be ascribed to errors of observation.

2.—If the stress at the center of gravity of one channel is above the average stress throughout the length of the column, and the corresponding stress for the other channel is similarly below the average stress, there must be an eccentricity in the application of the load at the two ends. If the stresses at the center of gravity of one channel member forms in the diagram a straight line which crosses the line of average stress, and that for the other channel crosses in the opposite way, the eccentricity of the load application must be oblique.

3.—If the stress at the center of gravity of a channel in near-by points is greater first in one channel and then in the other, the change may be due to crookedness of the column throughout that part of the length.

4.—If, in one channel or in one channel flange, the stress at the center of gravity remains constant and that of the extreme fiber varies, the change may be due to local crookedness of this channel and there will be a lateral bending of this member.

5.—If the front side of a channel has a higher stress than the back side, there must be bending action through its web, and *vice versa.*

6.—Changing stresses in the diagonally opposite corners of a channel may indicate twisting of the channel, and another combination of stresses may indicate a twisting or oblique distortion of the column as a whole.

An inspection of the diagrams shows that all these indications are found in the tests.

Does the Built-up Column Act as a Unit?—Engineers have often expressed doubt as to whether the parts of a built-up column act as a unit, although column formulas assume this unity of action. The tests throw some light on the question of the integrity of cross-section under load. The individual channel, of course, acts as a unit to resist bending action, though there are indications of twisting. The integrity of the whole section with reference to a plane parallel to the lacing seems probable, except as twisting action exists. With reference to a plane through the axis perpendicular to the plane of the lacing, this unity of action is not so certain. The tests on the distribution of compressive stress and likewise the cross-bending tests indicate that these built-up columns did not in all cases act as a unit but rather as two members not fully restrained by the lacing. The stresses in two channels at points in the same cross-section do not give the regularity of variation which would exist if the column bent as a unit. The elastic curve assumed by Column No. 1 under cross-bending load, shown in Fig. 2, Plate XII, differs from the computed elastic curve, though that for the wrought-iron column gives little difference. In the case of the posts of the White Heath Bridge, however, there is much closer agreement and a seemingly closer approach to unity of action.

Effect of Non-straightness of Built-up Columns Upon Distribution of Compressive Stress.—The effect of crookedness or other irregularities of a constituent member of a built-up column may be realized if a rough analysis of the case be made. Consider a part of one of the channels forming a column, taking the length between the connections of two adjacent lattice bars. This member is under compression. Owing to non-straightness or to the non-homogeneity of the material, the load on this short piece is not evenly distributed over the section;

that is, it is not centrally loaded, but may be considered to have an eccentricity with respect to the gravity axis. Call this eccentricity e (Fig. 19). Neglect any deflection of the piece under consideration due to the load. Call the compression load coming on this piece P; A its area of cross-section; I its moment of inertia about YY, and r the corresponding radius of gyration; and c the distance from YY

ECCENTRICITY IN CHANNEL MEMBER

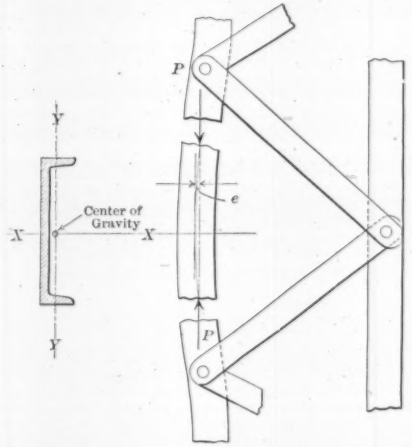


FIG. 19.

to the remotest fiber. Then the bending moment due to the eccentricity is Pe. The maximum stress will be

$$f = \frac{P}{A} + Pe\frac{c}{I} = \frac{P}{A}\left(1 + \frac{e\,c}{r^2}\right).$$

The excess of the stress in the extreme fiber of the piece over the average stress, produced by the eccentricity, e, is then $\frac{P}{A}\frac{ec}{r^2}$, and hence the term, $\frac{ec}{r^2}$, gives the proportionate excess of stress in the extreme fiber. This value is applicable to the channel, or to one flange of the channel, or it may be applied to the column as a whole by using the properties of the whole section. In the single channel under consideration, c is relatively large and r is relatively small, and the excess of maximum stress for a given eccentricity, e, may be expected

to be relatively large. It will be seen that for an excess of 50% in the extreme fiber of a channel of Column No. 1, *e*, by this formula, would be 0.045 in., and, in the wrought-iron columns, 0.057 in. A slight variation from straightness in a channel will account for considerable increase of stress.

Excess of Maximum Fiber Stress over Average Stress in Channel Members.—The diagrams and data show that the compressive stress is unevenly distributed over the cross-section of the columns tested, and also that there is great variation in this distribution at the various sections along the length of the column. It will be noted that in a number of sections the excess of stress was from 40 to 50 per cent. In one test of Column No. 1, an excess of 67% was found, and in the White Heath Bridge an excess of 73 per cent. Possibly these values were unusual, or the observations were erratic, but the indications of a fiber stress of from 40 to 50% in excess of the average stress were not uncommon.

It may be seen that among the causes to which the high fiber stress may be attributed are (*a*) non-straightness of the column as a whole, (*b*) non-straightness of the component channels, or eccentricity in the delivery of stress to them by the lacing, and (*c*) unknown eccentricity in the application of the load. It would be of interest to know how much of the increase of stress may be due to any one of these conditions. A study of the tests of Column No. 1 shows that generally only a small amount may be said to be due to non-straightness of the column as a whole. In but few cases is it found to be more than, say, 5 per cent. In four places it seems that the excess attributable to this may be estimated to be between 20 and 25 per cent. The effect of non-straightness of the individual channels seems to be greater. At several points the excess of stress attributable to this cause appears to be from 30 to 50 per cent. As already noted, a kink in the channels of 0.045 in. would give, by the analysis made, an eccentricity sufficient for a 50% increase in stress. Not all of this crookedness need be between adjacent rivet points, as the stress may not reach normal for some distance on either side. The effect of the third condition, eccentricity of application of the load, will vary with the construction. In Column No. 1 the effect of undetermined eccentricity of application of load appears to be not nearly as great as the effect of non-straightness of the component channels.

In the wrought-iron columns, which are much stockier, the lack of straightness in individual channels has less effect, seemingly less than 15%, and much the larger part of the high fiber stresses appears to be due to general column eccentricity or to eccentricity of loading.

The results for the posts of the White Heath Bridge are of interest in this respect. It is evident that the effect of non-straightness of channels was not very large, and also that the effect of non-straightness in the column as a whole was relatively small. There is, however, an evident bending in the direction of the web of the channels. For example, in U_3L_3 South, the back side of the channels has the maximum stress at the top and the front side at the bottom. The bending moment producing this may be due to obliquity of end pressures or to a bending by the connecting floor-beam and top chord. A twisting action is also apparent. Post U_3L_3 North gave quite similar results.

Compressive Strength of Lattice Bars.—In the discussion of stress developed in column lacing, the stress considered was the average over the bar. As usually attached, there is considerable flexure in the bar, and the ability of the bar to carry this eccentric load should be considered. The bars are most likely to fail in compression, since they act as long columns eccentrically loaded.

The tests of individual lattice bars (Fig. 17 and Table 12) show that the maximum fiber stress may be several times the average stress. It is also seen that even in a short lacing bar the maximum load carried is only about one-half the yield point of the material. The necessity of using very low working loads on lattice bars appears to be important. It will be noted that at low stresses there is similarity of distribution of stress in the slender bars and in the thicker bars, but the slender bars fail at smaller computed fiber stress.

The results of tests to destruction of individual lattice bars are fairly well represented by the formula:

$$\frac{P}{A} = 21\,400 - 45\frac{l}{r}$$

where P = load at failure, in pounds, A = area of cross-section, in square inches, l is the distance, in inches, from center to center of rivet holes, and r is the radius of gyration, in inches, of the cross-section of the lattice bar. These results may be considered to be applicable to "under" lattice bars. For "over" bars it seems probable that the average stress at failure would be considerably less.

Effect of Cover-Plates and End Connections.—In the tests of the White Heath Bridge, the effect of the cover-plate seems striking. The upper chord, U_3U_4, composed of two built-up channels with one cover-plate, gave an excess fiber stress of 20% at the worst section, while the posts, composed of two channels laced on both sides, gave a maximum of 73 per cent. The high value in the posts may be due to other causes, but it seems reasonable to expect that the cover-plate will act to reduce the irregularities in fabrication.

The connections of the ends of the posts evidently exerted a very noticeable effect on the stress distribution. In one of the posts tested, the stress was greatest at one corner of the post at the top and at the diagonally opposite corner at the bottom. It will be remembered that the posts were riveted to the top chord, and were connected with the lower chords by pins. The floor-beams were riveted to the sides of the posts, and this connection affects the stress distribution. Readings of deformations taken on the floor-beams and posts show that the loaded beam was partly restrained at the ends by the post and that there was an appreciable bending in the post.

Stresses in Column Lacing.—If the load carried by one channel of a column were the same throughout its length, no stress would be carried by the lattice bars. Such stress is developed whenever there is a change in the relative amount of loads carried by the two channels. If at the section, *AB* (Fig. 2), there is an equal division of load between the two channels, and also at the section, *CD*, and if at some section, *EF*, the division of load is unequal, it is evident that the lattice bars must be called into action to transmit this stress, and that transverse shear exists in the interval. In general, the conditions producing this must be complex, rendering analysis unsatisfactory, except in so far as the shear may be due to a known eccentricity of loading.

It is evident from the tests that the relative stress in the two channel members varies considerably from end to end and that the stress in the lattice bars also varies. It seems probable that the transverse shear developed may be traced largely to irregularities in outline, or at least that these irregularities may be expected to cover up other causes of stress in the lacing of centrally-loaded columns, if we include in such irregularities all unknown eccentricity.

The amount of transverse shear necessary to produce the maximum

observed lattice-bar stress (given in Table 6) is of interest, though of course it cannot be taken to be conclusive. The measurements were generally made at working loads. So far as observations were made on columns tested to failure, the distribution of stress remained much the same up to incipient failure. The values given in Table 6 indicate maximum average stresses in the bars such as would be caused by a transverse load ranging from 2 to 6% of the central compression load or of a transverse shear one-half as great.

DISCUSSION

Mr. Moses. JOHN C. MOSES, M. AM. SOC. C. E. (by letter).—This investigation is to be welcomed as a forerunner of many more in which examinations of the actual conditions under working loads will be made. Tests to destruction, or to any point beyond the elastic limit, are of very doubtful value, and the practice of making them the basis for designing has been justified only by the supposed impracticability of making such tests as are described by the authors. Certainly nothing can be more illogical than to find that load, the first application of which will cause destruction, and then to assume that any arbitrary fraction of that load can be carried indefinitely with a proportional factor of safety. The absurdities of this method have become especially apparent when applied to such materials as reinforced concrete, but are none the less existent when applied to homogeneous materials like steel.

The difficulty of making the proper kind of tests does not alter the facts, and the proofs now presented that such tests are not impracticable are very encouraging. They will need to be many in number and to be made on structures when first erected and again on the same structures after they have been in use a while. They will need especially to be made on details of all kinds in order to eliminate the very large element of guesswork which now enters into this very important part of designing. Some of the authors' conclusions bear very significantly on this subject. They find "no relation * * * between the stresses actually observed and the stresses computed by column formulas," and suggest that the design of details and conditions of manufacture may be such as to make void the usual assumptions entering into these formulas.

The whole subject of detailing is connected intimately with that of manufacturing. It is of essential importance economically, but has not received its proper share of attention. At present, working stresses are commonly limited to one-half the elastic limit, principally because of the uncertainty which exists in regard to the proportion of the strength of the main members, which can be developed by the details. With more exact knowledge of the actual distribution of stresses, and with the advance in the art of designing details which will accompany this advance in knowledge, it may become apparent that higher cost of manufacture will be much more than offset by reduced expense for material. Better methods of manufacture will also make possible the use of stronger steels, with all the possibilities such a change foreshadows. The recent rapid increase in the variety of sections rolled shows that this study of detailing need not be confined to the sections which have been standard for the past thirty years.

The authors show by example that a bend of less than $\frac{1}{16}$ in. in the channels of Column No. 1 would account for an increase of stress of

50 per cent. Shop experience shows that, aside from kinks due to handling during shipment from the mills or about the shops, the actual punching of angles, plates, and channels will curve the separate pieces sufficiently to cause such eccentricities as that cited, although the parts are apparently straightened by the lattice. Channels will not only bend, but will also twist, when punched in the flanges. Their flanges will perhaps be kinked at every hole by the blow of the punch. Plates will show buckles between the rivets after driving. Twisters must often be applied to columns during riveting to keep them from assuming a spiral form. Designers insist that lattice bars of I-shaped columns made of four angles, shall intersect on a common rivet, and that the tie-plates shall be of the same thickness as a single bar, expecting the angles to be pulled down by the riveting. They are pulled down, with a resulting kink in the outstanding legs as well as in the riveted leg. Numerous examples of this kind could be cited in support of the authors' statement as to conditions of manufacture. Mr. Moses.

The authors' experiments on lattice bars show the desirability of more adequate provision for staying the open sides of columns, when such columns are used; and the frequent bending of lattice bars in shipment and erection emphasizes the argument to those acquainted with the facts. A return to closed sections with separate joint pieces is among the possibilities.

The accumulation of further data is to be earnestly desired, and will certainly have a marked effect on the future development of structural work.

ARTHUR N. TALBOT, M. AM. SOC. C. E., and HERBERT F. MOORE, ESQ. (by letter).—It seems to the writers that the comments of Mr. Moses, which are from the viewpoint of the practicing engineer, are in accord with the results of the tests given in the paper. Many structural engineers have recognized that the shop and field processes used in the fabrication of structural members produce imperfections which give conditions varying widely from the ideal condition tacitly assumed in the analyses at the basis of the formulas used in design. How much the imperfections can be reduced, with better methods and greater care, and what conditions must be expected to be present under the best practicable methods of manufacture, can only be matters of conjecture. At different times, much interest has been manifested in questions connected with the action of the details of structural members while under stress; but, it may be doubted whether engineers generally appreciate the large variation in the distribution of stress over the sections commonly used in compression members. Messrs. Talbot and Moore.

The need of experimental work, to determine the stresses which are developed in compression members built under ordinary conditions of fabrication and erection, must be apparent. That it is actually practicable to make observations on structural members which will give

Messrs. Talbot and Moore.

trustworthy information seems evident from these tests. It is hoped that interest in this problem will be awakened among engineers and manufacturers, and that some institution or organization will take up the subject in a thorough and comprehensive manner. Such tests involve extreme care, and they are expensive, with regard to time and labor, whether done in the field or in the laboratory. A full study of the action of the compression piece at loads which do not stress the material beyond the elastic limit should be included. The expenditure involved is far beyond that of tests to destruction alone. It is not necessary to state that, to be of value, such an investigation must be accompanied by a careful study and analysis of the tests and results, though this has not always been appreciated. A programme of tests need not involve a large number of test pieces, but, to be really useful for the purpose in view, the time devoted to the test and study of each piece must be ample, and the total cost of even a fairly comprehensive investigation will be very large. It may be expected, however, that the value of the results would repay many times the cost of the work, and the expense would be justified by the added security and, perhaps, by the economy of metal which might result from the investigation.

Very recently the writers' attention was called to articles* by Professor Lilly, of Trinity College, Dublin, in which he shows, by experiments on compression pieces formed principally of rolled tubing, that, for columns having the same slenderness ratio, $\frac{l}{r}$, the strength varies with the relation which the thickness of the metal bears to the diameter of the tube, and he discusses a wrinkling action which, up to certain thicknesses, governs the strength of the column, irrespective of its length. The tests are quite suggestive. Professor Lilly did not investigate the loss of strength due to lack of straightness, to eccentricity of parts, and to the lacing together of component shapes to form one member. The conditions in built-up compression pieces make the irregularities of stress due to the form of the section, to the manner of connection of the component parts, and to the imperfections inherent in manufacture, even more marked than the variations to be found in simple rolled sections.

That the column formulas in common use have limitations, has been understood, but the effect which the condition of the component parts of a compression member exerts on the distribution of stress over the section has not been appreciated, nor has that of the eccentricty of connection of latticing, and of the possible non-integrity of section. It would seem quite probable that, for columns of the same length and containing the same amount of metal, one which is of stocky form and in which the metal is distributed so as to resist local flexural and tor-

* "The Strength of Columns," *Proceedings*, Inst. Mech. Engrs., June, 1905. "The Design of Struts," *Engineering* (London), January 10th, 1908.

Messrs. Talbot and Moore.

sional action will be much stronger and more satisfactory than a column of more flimsy form, which has its metal spread in thinner sections, even although the slenderness ratio, $\frac{l}{r}$, of the former may be considerably more than that of the latter. It seems reasonable to expect that a form of section which resists lateral bending, torsional, and collapsing stresses, will be much more satisfactory than a more flimsy type of column, for the lengths most common in ordinary bridge construction. For the longer lengths, the slenderness ratio must exert a stronger influence. As far as concerns the strength of the component angle, channel, or other structural shape used in a built-up compression piece, many engineers have been satisfied with the provision that the slenderness ratio of the component member may be less for the length between the points of attachment of lacing than the slenderness ratio for the column as a whole, and have given little attention to the possible non-integrity of the section, or to the probable effect of imperfections of manufacture. Fortunately, the large influence of the slenderness ratio in column formulas has given sections with which failures have not occurred. Whether a column formula should include a factor depending on the form of the section and the relative thickness of the metal, or whether the allowable stresses for any form of column should be based on experimental data for the section used, will depend on developments. In either event, full information as to the properties of various types of columns and various thicknesses of metal will be of great service. The same tests would determine the effect of the usual imperfections of manufacture. It would seem, too, that the reliability of any analysis of secondary stresses based on the assumptions of the common column formulas may be questioned. With reference to the effect of the form of column, Mr. Moses' suggestion in reference to the closed section and the form of lacing is pertinent at this time. His remarks on the imperfections in shop work and the frequency of their occurrence are also to the point.

It is evident that imperfections of manufacture need consideration in the design of columns. Whether better methods may be used, or whether the effect of necessary shop conditions shall be covered in the allowed working stresses, is worth discussion. Changes may be made so that lattice bars, although they take small stresses, may become more efficient compression members. Though column formulas have kept down the allowed working stresses, by reason of the large influence of the slenderness ratio, $\frac{l}{r}$, they seem (for short and medium lengths) to be deficient in putting so much weight on the slenderness ratio and so little on the shape and probable integrity of the cross-section.

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TRANSACTIONS

Paper No. 1125

IMPURITIES IN SAND FOR CONCRETE.

An Informal Discussion at the Annual Convention, July 8th, 1909.

BY MESSRS. SANFORD E. THOMPSON, R. W. LESLEY, C. P. HOWARD, RICHARD L. HUMPHREY, G. S. DAVISON, S. WHINERY, CHARLES M. MILLS, T. F. RICHARDSON, E. G. HAINES, AND SANFORD E. THOMPSON.

Mr. Thompson.

SANFORD E. THOMPSON, M. AM. SOC. C. E.—During the last two or three years attention has been called to several cases where concrete failed to harden, and these failures could not be attributed to the quality of the cement. Some of these cases have been investigated thoroughly, one in particular to a point where quite definite conclusions were reached, and this will serve as an illustration of at least one of the causes of poor concrete.

It has been proved conclusively that the sizes, and the gradations of sizes of particles of sand, affect the density, strength, and permeability of the mortar, and definite laws governing these relations have been framed, notably by Mr. Feret,* in France, which show that with sand having grains of known sizes the strength of the mortar may be estimated. These laws have been corroborated in a general way by tests in the United States and elsewhere.

In the United States, also, further tests have shown the effect of scientific methods of proportioning or grading the aggregates of the concrete upon its density and strength.†

However, one frequently finds a sand which absolutely fails to obey the laws of gradation of sizes, or of density. Not only may the

* *Annales des Ponts et Chaussées*, IV, 1892.

† "Laws of Proportioning Concrete," by William B. Fuller and Sanford E. Thompson, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIX, p. 67.

mortar or concrete made from such sand fail to show the strength which would be expected, but it actually fails to harden, or the hardening which should take place within a week is delayed for perhaps two or three months. Mr. Thompson.

Frequently, the sand which produces such bad results looks all right, and even those who have had much experience in concrete construction may be deceived by its appearance. Its mechanical analysis, that is, the gradation of sizes, may be good, and it may appear clean, and yet the quality may be such that it has to be absolutely prohibited from use in concrete, and, if used without previous testing, it will cause failure.

This does not disprove the laws referred to, but simply indicates that there is something further; that the laws of density apply to a clean sand, and that frequently it contains some other material which affects its combination with cement, either mechanically or chemically, or perhaps both. Such results, such failures of the mortar and concrete to set, show the absolute necessity, not merely of a careful examination of a sand which is to be used, not merely of a mechanical analysis to determine the sizes of the particles, but of a laboratory test, as thorough a test, in fact, as would be given to the cement itself.

In the special case referred to, a two-story machine-shop with concrete walls was being erected by local contractors, with no engineering advice except in the preparation of outline plans. The walls of the basement were 12 in. thick, those of the first story, 10 in. thick, and those of the second story, 8 in. The interior of the building was mill construction with timber beams and plank floors. One night, the walls being just above the level of the second floor, during a severe windstorm the building collapsed. An examination by the speaker, who was called on to make an investigation, immediately after the accident, showed that the concrete, although mixed in the proportions 1 : 2½ : 5, had failed to harden, even in the basement.

The concrete of the entire building was so soft that, although parts of it had been laid for at least two months, a knife blade could be thrust into it, and it was even difficult to pick from the wall a piece hard enough to carry away as a sample. The appearance of the concrete was dark and dirty. There was a thin, hard skin on the outside, which had helped to deceive the contractors into believing that the material would eventually harden, although they were by no means satisfied with it.

Investigation showed that the concrete had been proportioned 1 : 2½ : 5, and mixed in a satisfactory manner. It was hand-mixed, but apparently well done, and tests of the actual proportions by analysis were considered unnecessary.

It was evident that the cause of the trouble lay in the materials. The last of the carload of cement had been used, only the empty bags

Mr. Thompson.

remaining; and because representative samples of this cement could not be obtained, the other materials had to be tested much more carefully than otherwise would have been necessary.

The sand and gravel were taken from the site of the building when the cellar was excavated. The gravel was ordinary New England gravel, ranging from fine, that is ¼-in., up to particles perhaps 2 in. in diameter. It was fairly clean. Some of the pieces were slightly coated with dirt, but not more so than is almost always found in a gravel bank.

The gravel screened contained about 12% of sand finer than the ¼-in. size, that is, the screening was imperfect. However, this is almost always the case with screened gravel, so that in gravel concrete there is apt to be an excess of sand over the nominal proportions specified. In screening, a part of the sand, especially on wet days, is carried down with the larger stones.

The gravel was washed as gravel is usually washed in hand-mixed concrete. A hose was turned on the pile before the mixing was begun, and the fine material was washed down to the bottom of the pile and shoveled into a wheel-barrow with the rest. Then the gravel from the barrow was dumped on the mixing platform, and was again washed with the hose, the dirt simply flowing with the water from the surface to the bottom of the pile to be shoveled up and mixed with the gravel in the concrete. This is the ordinary but ineffective way of washing gravel in hand-mixing, unless special apparatus for washing is used.

The sand appeared to be good. In some places it was rather dark colored, but the most of it would pass an ordinary inspection, and would be called a sand of fairly good quality, certainly good enough for the work which was being done. The results of the mechanical analysis of the sand, given in Table 1, were above the average. Three per cent. by weight passed a No. 100 sieve, and about 25% was caught on a No. 8 sieve, but a closer examination of the bank showed considerable variation in the sand; in some places it was rather dark and reddish in color, while piles which had dried out looked "dead."

The representative of the cement company suggested a practical test which had never before come to the speaker's attention. Taking a double handful of moist sand from the pile, he allowed it to run between his hands as they were held with the thumbs up about 1 in. apart, at the same time moving his hands back and forth. Repeating this operation several times, always taking naturally moist sand from the interior of the bank, between the fingers of both hands there was collected a dark slimy substance which contained scarcely any grit. Some of this, scraped from the fingers and afterward tested by ignition, was found to consist almost entirely of vegetable matter.

A further examination of the bank showed that, in places where it had been cut to a vertical face for an excavation or for screening out

the sand, the rain had washed down from the surface soil a material similar to that which had collected between the fingers in the foregoing test, which formed a scum on the vertical face of the bank. Mr. Thompson.

The surface soil was an ordinary, medium quality of loam, with, however, one or two dark, almost black, streaks in it, from ½ in. to ¾ in. thick.

All indications seemed to point to the cause of the trouble being vegetable matter in the sand, and vegetable matter which had apparently been washed down by the rains from the surface soil into the porous sand and gravel underneath, gradually coating the grains.

Tests of Sand.—Samples of the sand and stone were subjected to thorough tests. The mechanical analysis is shown in full in Table 1.

TABLE 1.—MECHANICAL ANALYSIS.

Analysis No.....	A-92.			A-96.		
Description	Average sand from excavation.			Reservoir sand.		
Date of collection..	May 5th, 1908.			1907.		
" " analysis ...	May 8th, 1908.			May 9th, 1908.		
Percentage of moisture.....	2.6 before drying.			0		
Size of sieve.	Total weight passing.	Total percentage passing.	Percentage passing ¼-in. sieve.	Total weight passing.	Total percentage passing.	Percentage passing ¼-in. sieve.
Inches. / No.						
1.50 / 1½ in.	97.0	100.0	100.0	100.0	100.0	100.0
1.00 / 1 in.	97.0	100.0	100.0	100.0	100.0	100.0
0.50 / ½ in	97.0	100.0	100.0	100.0	100.0	100.0
0.25 / ¼ in.	92.7	95.6	100.0	96.2	96.2	100.0
0.16 / 5 in	81.8	84.4	88.4	88.9	88.9	92.5
0.0583 / 12 in.	56.5	58.2	61.0	68.2	68.2	71.0
0.0335 / 20 in.	39.5	40.7	42.6	48.2	48.2	50.1
0.0148 / 40 in.	14.6	15.0	15.8	17.0	17.0	17.7
0.0055 / 100 in.	2.6	2.7	2.8	2.3	2.3	2.4
0.0030 / 200 in.	1.8	1.9	2.0	1.1	1.1	1.1

Very fortunately, for purposes of comparison, the speaker happened to have in his laboratory a large sample of another sand which had been used satisfactorily in the construction of a reservoir* near Boston. This reservoir, because of its size and shape—100 ft. in diameter and nearly 50 ft. high—required special tests of the materials. The mechanical analysis of this good reservoir sand, shown in Table 1, was found to be almost identical with the sand used in the building which failed; moreover, the chemical composition was also practically identical, each containing about 75% of quartz.

Notwithstanding this apparent similarity, 1:3 mortar from the reservoir sand gave a strength of 272 lb. in 7 days and 332 lb. in 28

* *The Engineering Record*, January 12th, 1907, p. 32.

Mr. Thompson.

days, while the poorest sample of the sand in question gave an average of 20 lb. in 7 days and 75 lb. in 28 days.

Volumetric Test.—To be sure that the bad quality of the sand was not due to the shape of the grains, which might prevent proper compacting, and, consequently, an excess of voids in the mortar, volumetric tests were made, and the density was found to be normal. The density of 1:3 mortar made with the sand in question was 0.679, and that of mortar made from the reservoir sand in the same proportion was 0.683.

Mica.—A little mica was observable in the samples. A careful examination, however, indicated that this was not sufficient to cause trouble.

Clay.—It has been sometimes claimed that, from a chemical standpoint, clay matter is injurious to sand. An examination of the chemical analysis showed, however, that the amount of clay, although large, was approximately the same as in the good reservoir sand, and, therefore, could not be considered as the cause of the poor quality.

Tensile Tests.—The tensile strength of the mortar from the sand in question, as already stated, averaged 20 lb. per sq. in. in 7 days and 75 lb. in 28 days.

The tests in different series using sand from different parts of the bank ranged, in the 7-day test, from 11 to 40 lb. To be sure that the cement used in testing or the laboratory methods were not at fault, tests were made in two different laboratories and with three different cements. Three different proportions were also used, namely, 1:2½, 1:3 and 1:3½. Specimens, for comparative tests in air, were stored in a moist closet, and in water. The results all checked so closely as to eliminate the question of cement or of manipulation.

Comparison of Moist and Dry Sand.—A comparison of 1:3 mortar made with the moist sand as it came from the natural bank and with the same sand after drying, showed some relative increase in strength due to the drying. In this series the average strength at 7 days of the sand direct from the bank was 8 lb., and of the dried sand mortar 23 lb.

Comparison of Natural Sand and the Same Sand after Washing.—A comparison of the sand in question before and after washing was as follows:

Unwashed.....	40 lb. at 7 days,	102 lb. at 28 days.
Washed.....	97 " " 7 "	196 " " 28 "
Standard	200 " " 7 "	275 " " 28 "

The grains of the washed sand were not thoroughly clean.

Microscopical Examination.—The microscopical examination of the grains of sand as obtained from the bank showed them to be covered with a dark brown coating which did not readily brush or wash off. The particles of the Waltham sand, on the other hand, were clean. A strong magnifying glass was found efficient in examining the grains for coating.

Washing Tests.—To examine and test the character of the silt in the sand, it was found necessary to remove the fine matter by washing. Mr. Thompson.

To compare the results by washing and by screening out the silt through a No. 100 sieve, tests were made in both ways with the following results:

Worst sample of defective sand: 1.60% silt by washing; 1.14% by screening.

Average sample of defective sand: 2.46% silt by washing; 2.68% by screening.

Reservoir sand: 1.16% silt by washing; 3.66% by screening.

Although there is but slight difference between the results from washing and from screening, the chemical analyses indicate that the washing removes much more of the deleterious vegetable matter than the screening, so that this process should be followed when testing silt in sand.

It is noticeable in the above results that the poor sand gave a larger percentage of silt by washing than by screening, while the good sand gave a much smaller percentage by washing, indicating that in the latter case a large proportion of the very fine material was of heavier mineral origin.

The method used in washing to remove the silt was a very simple one, but was found to be more effective than several other more elaborate methods which were tried. The sand was placed in a large-mouthed quart bottle about half full of water, and shaken thoroughly. The dirty water was poured off, and the operation repeated several times until the water was nearly clear. The wash-water was then evaporated, the residue thoroughly dried, and the loss on ignition, which determined the quantity of organic matter, was found.

Before igniting, the silt was passed through a No. 100 sieve to remove large particles of dirt which evidently would not be injurious if distributed through the mortar.

Chemical Analysis of Silt.—Both the poor sand in question and the good reservoir sand were washed, and a chemical analysis was made of the silt, including a test of the residue by ignition. The analyses of the silt were as shown in Table 2.

TABLE 2.—ANALYSES OF SILT.

	Percentage in reservoir sand.	Percentage in average sand, B, from site.	Percentage in worst sand, G, from site.
Moisture	7.80	8.24	13.07
Silica	54.12	45.32	34.94
Alumina	23.10	25.13	20.80
Oxide of iron	9.00	6.71	5.90
Lime	2.07	1.93	.....
Organic matter	2.50	11.86	26.33
Approximate percentage of clay	45.0	50.0	35.0

Mr. Thompson. Inspection of these analyses shows but one notable difference: The organic matter in the silt from the reservoir sand was 2.50%, while in the silt from the poor sand it was very high. In one of the samples of poor sand the organic matter was 11.9% and in the other 26.3 per cent. When ignited, the organic matter in the poor sands gave off a peculiar odor of woody fiber, or leaf mould, thus indicating its probable vegetable origin. The good reservoir sand, on the other hand, gave no appreciable odor, and showed a reasonably small percentage of organic matter.

Reference has been made to the comparison of screening and washing. Tests by ignition show that, by washing, frequently double the percentage of organic matter is obtained than by simply screening.

It is noticeable that the silica is low, but it must be remembered that this is the analysis of the silt and not of the total sand.

Introducing Silt Into Standard Mortar.—To confirm still further the conclusion that the silt was the cause of the trouble, a mortar was made up of 1:3 standard sand with an addition of 1½% of silt based on the weight of the sand. The resulting tensile strength of the mortar was 29 lb. per sq. in. in 7 days and 107 lb. in 28 days, whereas the normal strength of the standard sand mortar was about 200 lb. in 7 days and 275 lb. in 28 days.

A similar test was made by introducing 1½% of silt into neat cement, and the resulting strength was about one-half that of the neat cement without silt.

Concrete Tests.—Specimens of concrete were also made, sand from different parts of the bank being used in different tests. The cement used in these tests was of a well-known brand, which was carefully tested to see that it was normal. The poor sand and gravel from the site of the building were used in the test as well as the good reservoir sand and gravel. The blocks were broken at the age of about 18 days at the Watertown Arsenal, with the results shown in Table 3.

Hardening of Specimens.—Careful watch was kept of the specimens of concrete and of the mortar as they hardened, and pats of the concrete were also made. The mortar and concrete made with the good reservoir sand set up hard within 24 hours with a light gray color, while the other specimens remained so soft that they could not bear the pressure of the thumb nail for several days.

Conclusions.—As already stated, the tests indicated conclusively that the trouble with the sand was due to the vegetable matter which it contained, and that subsequent tests of other sand, and examinations of structures, indicate this to be a common cause for poor mortar or concrete. In many cases no failure results, but the concrete does not harden properly and never becomes as strong as it should.

The percentage of silt given in the chemical analysis of the silt appears to be large, being 11.9% in one sample and 26.3% in another.

When given in terms of the total sand, however, it is a very small percentage, because there is so small an amount of silt in the sand. Based on the weight of the total sand before washing, therefore, the percentage of silt as it comes from the bank is 0.27% in one case and 0.39% in another, an extremely small amount, and one which would not be indicated by any test of settlement in water or rubbing in the palm of the hand. Mr. Thompson.

TABLE 3.—COMPRESSIVE STRENGTH OF CONCRETE, IN POUNDS PER SQUARE INCH.

Age, 18 days. Proportions, 1 : 2½ : 5.

Aggregate.	Cement K.		Cement A.	
	Individual.	Average.	Individual.	Average.
Sand A. .....and Gravel E.	....	....	200	200*
" ¼ A + ¾ B " " E	818	....	625	....
" ¼ A + ¾ B " " E.....	692	755	749	687
" W..... " " E.....	....	....	653†	....
" W..... " " W.....	....	....	1 505	1 505*

NOTES.—Sand A.—Worst sand from site of building.
Sand B.—Average sand from site of building.
Sand W.—Good reservoir sand.
Gravel E.—Screened gravel from site of building.
Gravel W.—Screened reservoir gravel.
* Only one specimen, but a good one.
† Only one specimen, and possibly slightly defective.

These and other tests indicate that there are two percentages of vegetable matter, which appreciably affect the quality of a sand: First, the percentage of vegetable matter in the silt, and second, the percentage of vegetable matter in the sand. Although the tests made thus far are too few to enable one to draw definite quantitative conclusions, it would appear that, in order to be injurious, the organic matter in the silt must be more than 10%, and, at the same time, the organic matter in the total sand must be more than one-tenth of 1 per cent.

Both these conditions are necessary, because it appears from tests that, in certain cases, the vegetable matter in the silt may be greater than 10%, but there may be so little silt in the sand that the organic matter in the total sand will be less than one-tenth of 1%, and the sand will pass the tensile test.

Whether the cause of the results is due entirely to chemical action or whether it may be due in part to mechanical action, the organic matter surrounding the grains so that the cement will not adhere, has not been determined. Most probably the cause is chiefly chemical, but in a small measure also mechanical. The subject appears to be of sufficient importance to warrant further and thorough investigations.

Mr. Thompson. In conclusion, special stress should be placed on the necessity, in concrete work, of thorough tests of the sand. It is not merely necessary to examine sand with the eye; it is not sufficient to test it by rubbing it in the hands; it is not enough to make a mechanical analysis and to determine the sizes and the gradations of the particles, but, in every case, unless the sand is from a bank of known good quality and has been previously tested, as careful tests as are required of the cement should be made. Probably the best one is the ordinary tensile test advocated by the Society's Special Committee on Concrete and Reinforced Concrete, which requires that:

"Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand."

In case a sand must be used immediately, with no time to make tensile tests, or in case special investigations are needed to determine the causes of poor quality, the washing test and determination of the organic matter are of special value. The mechanical analysis, which shows the proportions of the grains of different size, is also of great value as indicating the comparative value of different sands which are free from organic matter.

There is one further thought which might be brought out. These impurities have been spoken of as dirt and matter which might be washed out. One sample of sand which was sent to the speaker from Philadelphia, had a great deal of dirt in it, that is, it had much more fine material than the defective sands which were investigated, and yet washing it increased the strength of the mortar instead of decreasing it. The fine material was found to be, not organic matter, but of mineral composition.

Mr. Lesley. R. W. Lesley, Assoc. Am. Soc. C. E.—Mr. Thompson's contribution is very interesting. In the light of the recent report of the Special Committee on Concrete and Reinforced Concrete, of this Society, it rather emphasizes the principle of "locking the stable door after the horse has been stolen;" in other words, the building had fallen before the exhaustive examinations of sand were required. On the other hand, the speaker believes that the report contains, in one paragraph, what is more germane to the subject of concrete and reinforced concrete than any other lesson, namely, that the sand shall be tested in the same way that the cement is tested. The mortar must be a standard mortar. This report provides a means to determine that question.

At the present moment there is no doubt that in every large cement manufacturing plant in the United States there are many inspectors who are testing cement prior to its shipment to various parts of the

country. The speaker happens to know that, to-day, in one mill, there are no less than twenty or twenty-five men engaged in testing and inspecting cement. One part of that cement is to be used to three parts of sand, and yet the speaker does not know of anyone anywhere who is testing the sand to be used in the same manner that the cement is being tested. Mr. Lesley.

The engineer requires that cement shall be tested in a dozen different ways, to show that it meets the requirements of the specification, and no cement manufacturer, under existing conditions of inspection at the mill, can produce a cement that will not be actually and scientifically right; and yet that cement may be used, in the proportion of one to three, with a sand that no one has tested, no one has examined, and about which nothing is known.

Mr. Thompson's paper brings us face to face with a state of affairs which has been going on for many years, and, by making a requirement that all sand shall be tested in the same manner that cement is tested, the Committee on Concrete and Reinforced Concrete has certainly taken a great step forward.

In the recent report of the Building Code Commission of the Board of Aldermen of New York City, a step farther has been taken. Whereas, to-day, cement only is tested, under the report of the Committee on Concrete and Reinforced Concrete, both the cement and the sand will be tested. But there is still another step: Suppose one has good cement and good sand, and yet the building fails. That failure may be due to the fact that the contractor has used seven or eight, or even ten parts of sand, and again there is an opportunity "to lock the stable door after the horse has been stolen." In this particular report of the Building Code Commission, it is provided that the sands represented in the mortar shall be inspected from time to time, and that the mortar from the mortar box shall have 70% of the strength of the same cement, used with standard Ottawa sand. In other words, it places the responsibility for the mortar which binds the concrete, or which binds the masonry, on the men who are actually providing that mortar.

The cement is to be tested, the sand is to be tested, and the mortar is to be tested, and the speaker believes that this is a most important step forward. He also believes that, this country being one of the most progressive in the use of concrete, the day is not far distant when the American, instead of whittling timber. with a knife, will whittle concrete with a shovel; that the same cleverness which, heretofore, has enabled American genius to make its way forward, will enable it in the future to whittle with a shovel, and to do things with this plastic material which to-day seem impossible.

Recently, at a meeting of one of the scientific societies, one man read a paper on the use of concrete with nails as reinforcement.

Mr. Lesley. Another engineer stated that he had made some concrete and had reinforced it with jackstones; and that statement was again made ridiculous by a man who had used old hoop-skirts as a reinforcement, and if American genius has gone so far in the use of concrete, there seems to be no limit to what it will do in the future.

Mr. Howard. C. P. HOWARD, M. AM. SOC. C. E.—The speaker has recently seen some specifications in which there was a provision that the stone used should be free from oil, and understood that the parties using these specifications went so far as to refuse stone from certain quarries on account of the oil which it contained. Such a precaution would probably not occur to most engineers. It emphasizes the importance of the most careful examination of all three ingredients, namely, cement, sand, and stone, and also of actual tests of concrete briquettes made from the same materials used in the structure.

Mr. Humphrey. RICHARD L. HUMPHREY, M. AM. SOC. C. E.—There has been a decided change in the requirements for sand, and this is necessary, because, to-day, the character of the sand used in cements, mortars, and concretes is of the highest importance since it has a vital effect on their strength. Specifications which require the sand to be clean and sharp are wholly inadequate, and yet there are many engineers who pass on the quality of sand by its appearance, thus complying with this specification. The fallacy of thus judging it is readily illustrated in an examination of sands from different parts of the country, such an examination showing how easily the eye can be deceived as to its quality. Sands have been sent to the speaker from various parts of the country, and he has had the opportunity, in a number of cases, of noting how easily it is possible to be deceived. As an instance, two sands from the Isthmian Canal may be given. The one which complied with the specification by being clean and sharp, and apparently the better of the two, proved by test and careful examination to be inferior to the sand which was not sharp and appeared to be dirty. The physical tests were the means of determining the relative quality of these two sands. As a matter of fact, the poorer sand was the cheaper as it was more readily accessible for the work in question. This, the speaker believes, is one of the many illustrations of the value of making physical tests of sand before passing on its quality. Elaborate tests of cement are made, and just as thorough tests of the sand should be made, because its effect on the strength of concrete is as great as, if not greater than, the cement which binds it together. Many sands which appear to be sharp and of a fine quality are of uniform size and contain so much fine material that they produce very slow-hardening mortars, thus interfering with the stability of the structure in which they are used.

There is no doubt that a great deal of the inferior sand used to-day is the cause of failures. It may be stated that the use of this sand

Mr. Humphrey.

is in a large measure due to a reasonable ignorance on the part of engineers and others in charge of the work. It is highly important that the engineer should be alive to the necessity of testing the sand just as rigidly as he tests the cement, thereby securing a better grade and eliminating many troubles which may arise from the use of improper material. The report of the Society's Special Committee on Concrete and Reinforced Concrete, in requiring a mortar-box test, marks an important step forward.

The fine material found in sand, which is generally referred to as loam, should be properly called silt, because it generally consists of very finely-divided material of the same character as the sand itself. In the investigation of sands from all parts of the country, by the Structural Materials Testing Laboratories of the United States Geological Survey, it has been found that the presence of this fine material, if it is of a granular character, proved beneficial, as it increased the density of the mortar or concrete, and consequently its strength. In an effort to increase the impermeability of mortars and concretes, so-called colloidal clays have been used. A fine sand is always used with the idea of filling voids, and the speaker believes that its character is immaterial, if it is granular and is sufficiently fine to fill the voids properly. Indeed, where the so-called colloidal clays seem to produce a greater density, it has been found that a very finely-divided sand would produce the same effect. On the other hand, there are sands and gravels which contain an excess of finely-divided material which it is necessary to screen out in order to get the requisite density and strength. In the study of sands referred to, an appreciable quantity of organic material is rarely found. Where sand or gravel is not well graded, or concrete is not properly proportioned to secure maximum density, a finely-divided material or a little extra cement itself may be added with beneficial effect. Where the sand or gravel naturally possesses the requisite amount of fine material to fill the voids, the addition of more material of this character would tend to reduce the strength of the mortar and prove otherwise detrimental. Again, a large percentage of finely-divided material will produce mortars and concrete which set very slowly, and this is a decided disadvantage. In this discussion, material finer than will pass a No. 200 sieve is referred to as silt. Many natural sands contain 6 or 8% of a very fine material, which, by reason of filling the voids, adds materially to the strength of the resulting mortar and concrete, and gives rise to the impression that small percentages of finely-divided material are beneficial to the mortar or concrete. If the mortar or concrete is granular, and the fine material does not occur as a coating on the particles of sand, and is not in too great excess, there is no doubt that it will be beneficial. On the other hand, fine material which is of a flaky character and coats the particles of sand or gravel, has a decided weakening action, and is objectionable.

Mr. Humphrey. The use of clays of various kinds is also objectionable, because in order to mix them with the cement properly, they should be thoroughly ground and dried to an impalpable powder. Otherwise, they will have a tendency to ball up, and the result will be little nodules of clay in the mortar or concrete.

Mr. Davison. G. S. DAVISON, M. AM. SOC. C. E.—The importance of using proper sand for cement mortar has long been recognized. Some fifteen or sixteen years ago the speaker drew up specifications for cement mortar to be used in the foundations of a large industrial establishment, and great care was taken to specify that the sand should be sharp river sand, such as is found in abundance in the neighborhood of Pittsburg.

This sand was shipped to the point of construction by car, and at considerable expense to the contractor for freight. After a few days it was discovered that the mortar was not setting properly, and, on making a series of investigations, it was decided that the trouble was caused by the sand. It was just as specified, but the mortar made from it would not stand the tests. As a temporary expedient, it was agreed that, for a few days, and in some unimportant foundations of the building, the contractor would be permitted to use, in the mortar, bank sand which was being excavated for the foundations for the building. Much to everybody's surprise, the mortar made with this bank sand was satisfactory in every particular.

Mr. Whinery. S. WHINERY, M. AM. SOC. C. E.—The word "sharpness" is frequently used in speaking of the qualities of sand, and perhaps it is worth while to call attention once more to the fact that it is an indefinite and even deceptive term. This, the speaker believes, has been pointed out a number of times in the *Transactions* of the Society. The test usually applied to determine the sharpness, that of grinding the sand between the fingers, is, to say the least, not decisive. The rounded and polished grains of a beach sand, if of a certain size, seem by this test to be decidedly sharp. If these same grains be crushed to a comparatively fine powder and rubbed between the fingers, they will give the impression that they lack sharpness, although the individual grains are angular, with very sharp edges.

It is undoubtedly true that the character of the different kinds of sand has not been investigated sufficiently in connection with the making of mortar and concrete. There may be great differences in the character of sands, which, in most respects, appear to be alike. Some have a greater degree of what has been called the quality of absorption, and the cement seems to take a stronger hold of them than it does of other sands which are apparently just as good. This quality of the surface of sand grains deserves more careful investigation than it has yet received. Until engineers are able to drop the indefinite term "sharpness," in specifications for sand, and to substitute for it some accurate scientific description of the sand wanted, unsatisfactory results may be expected sometimes.

Mr. Mills.

CHARLES M. MILLS, M. AM. SOC. C. E.—One of the sands tested by Mr. Thompson in his studies subsequent to the special investigation to which he refers was taken from the excavation for the subway on Market Street, Philadelphia, Pa., from the intermediate and lower strata, the cleanest being selected for the work.

The large proportion of silt was regarded with suspicion, and its use in concrete, except in unimportant details, resulted only after thorough tests, extending over 90 days. The material was hauled to a yard, passed through ¾-in. screens, and used without washing; it contained a good proportion of coarse particles, well graded from coarse to fine. Daily tests were made, in order to control the quality, the proportion of silt varying from 6 to 29 or 30%, which was determined by the matter not settling in 45 sec. when a sample was agitated in water, the sample being successively agitated in fresh water until the water was clear. The proportion of silt averaged about 21½%, the upper limit of 30% being unusual, and representing the approximate limit for rejection. The silt was practically impalpable, and analysis showed that it consisted usually of fine sand and disintegrated mineral matter in about equal proportions. The sample furnished to Mr. Thompson contained 0.4% of organic matter which did not seem to affect the strength in mortar briquettes.

Briquettes in the proportion of 1:3 gave better tensile strengths than those obtained from any other of the coarse sands or fine gravels offered for use, except in the case of a certain river-washed silicious sand which was remarkably clean and well graded from coarse to fine. With this exception, the tensile strength of 1:3 briquettes made with the excavated gravel was in excess of that obtained from briquettes made with all other materials, except mixtures of stone dust and stone grit. A few typical tensile tests, given in Table 4, show the results with standard quartz sand, the river-washed sand referred to, and the excavated gravel, in which the content of silt is stated.

The tests in Table 4 are in general agreement with many others made during the progress of the work, both in the values obtained and in the vagaries with respect to the effect of the varying proportions of silt, and, in considering the latter, the constituents of the silt should be borne in mind.

The silt comprised the fine material so desirable in producing dense and water-tight concrete, and, at the same time, the material in which it occurred made tenacious mortars.

The tests on 6-in. cubes revealed the same vagaries with respect to the effect of more or less silt in the excavated gravel, the tests and experience with the concrete in which this gravel was used confirming the favorable conclusions reached from the tensile tests respecting the value of the gravel for use as the fine aggregate. The presence of so much fine material made it necessary to exercise special care in

TABLE 4.—TESTS ON STANDARD BRIQUETTES, MADE WITH PORTLAND CEMENT, IN PROPORTIONS 1:3.

Material.	7 days.	28 days.	3 months.	6 months.	Maximum silt. %	Minimum silt. %	Average silt. %	Remarks.
Standard quartz sand.....	(33) 213	(33) 278	(27) 327	(18) 320	.....	.....	.....	Values given are averages from the number of briquettes in parentheses.
Excavated gravel.....	(33) 291	(33) 364	(27) 427	(18) 475	29.00	6.80	18.30	
River-washed gravel.....	(12) 276	(12) 405	(12) 472	(6) 443	4.00	Trace.	1.62	
Excavated gravel.....	(3) 297	(3) 396	(3) 401	(3) 451	29.00	.....	.....	Values given are averages from three briquettes.
Excavated gravel.....	(3) 311	(3) 393	(3) 453	(3) 469	.....	6.80	.....	
River-washed gravel.....	(3) 311	(3) 441	(3) 439	(3) 439	4.00	.....	.....	
River-washed gravel.....	(3) 254	(3) 413	(3) 517	(3) 447	.....	Trace.	.....	

Mr. Mills.

cleansing the scum from the surfaces of previously deposited concrete in making joints. Briquettes made with cement and the pulverized dried scum gave very small tensile strengths. Mr. Mills.

It is desired, incidentally, to emphasize the necessity for the examination of the properties of silt in any given case, since it may either be a source of dangerous weakness in mortar, or a valuable constituent in gravel for concrete, when water-tightness is desired.

One purpose in introducing the foregoing is to invite a discussion of the following question, which is asked with full appreciation of the laboratory researches and the experiments with colloidal clay and other materials in reducing the permeability of concrete: When water-tightness, or the reduction of permeability in concrete is necessary, and the available materials for concrete do not contain in themselves a sufficient quantity of the fine material required for this result, is it wise to introduce a fine material in the intermediate aggregate, or is it better to increase the quantity of cement?

In considering the foregoing, it is desired that the practical features of the case be considered from the standpoint of the engineer who aims to control the quality of the materials, and to see that such materials are properly used. It is also understood that the most judicious proportioning of the coarse and fine aggregates will be made in any given case and that the proportion of cement will produce the desired structural strength. It is obvious that local considerations of available supplies will affect the decision, but it is desired to assume that the requisite materials are available at reasonable, or what may be termed balanced, prices. It is also to be assumed that the materials are mixed by machine, according to approved methods, and that the question is intended to apply to natural or ordinary commercial material, and not to patented or secret compounds.

T. F. RICHARDSON, M. AM. SOC. C. E.—The statement has been made that fine sand or silt tends to increase the density of mortar. This may be true for lean mortars where 1 part of cement is used to 4 or 5 parts of sand, but the speaker does not think that it is true for richer mortars such as are commonly used in American practice. It certainly is not true for mortar of 1 part of Portland cement to 2 parts of sand. Mr. Richardson.

During the construction of the Wachusett Dam of the Massachusetts Metropolitan Water-Works, extensive experiments were made to determine the effect of using sands of different sized grains, and various combinations of sand, on percolation and on the tensile strength of mortars. Three grades of sand, called fine, medium, and coarse, were used in the experiments; the fine sand all passed a No. 100 sieve, the medium sand all passed a No. 30 sieve and was held on a No. 100 sieve, and the coarse sand all passed a No. 8 sieve and was held on a No. 30 sieve, these sieves having 10 000, 900, and 64 meshes, respectively, to a square inch. All three sands had practically

Mr. Richardson.

the same percentages of voids, ranging from 41 to 42%, and the fine sand had practically the same percentage of voids and the same mechanical analysis as the cements used in the experiments, which were standard American cements.

Experiments to determine the percolation through, and the tensile strength of, the mortars, were made with the fine, medium, and coarse sands, and with nineteen mixtures of these sands, the mortars experimented on consisting of 1 part of cement to 1, 2, 3, and 4 parts of sand by measure. A mixture of 75% of coarse sand and 25% of fine sand gave a sand having 31% of voids.

As illustrating the density of the mortars, it was found that a 2:1 mortar, made from two barrels of coarse sand and one barrel of cement, gave 2.4 barrels of mortar, while, if two barrels of fine sand were used, 3.0 barrels of mortar were obtained. All mixtures of the fine, medium, and coarse sands gave more mortar than the coarse sand, the 75% of coarse and 25% of fine sands giving 2.53 barrels of mortar for the 2:1 mixture. This was to be expected, as the 41% of voids in the coarse sand, being large, were filled by the cement, while the 42% of voids in the fine sand were not filled by the cement, as the fine sand and the cement had grains practically of the same size.

In testing for percolation and for the tensile strength of the mortars, it was found that mortars of coarse sand with no admixture of fine or medium sands gave practically the least percolation and the greatest tensile strength, with 1 part cement to 1 part or 2 parts sand. For mortars of 1 part of cement to 3 parts of sand, the mortar made of 50% coarse, 25% medium, and 25% fine sands gave slightly better results than that made with 100% of coarse sand, both as to percolation and tensile strength, and this was also true to an increasing extent for mortar of 1 part of cement to 4 parts of sand. The mortars of fine sand, whatever the proportions of cement might be, in all cases gave the greatest percolation and the least tensile strength.

The sands used in the above experiments were almost entirely silica, and contained little or no alumina. The pressure was 75 lb. per sq. in.—equivalent to a head of about 170 ft.

In coarse sand the particles themselves are impervious to water, and the fine cement fills the interstices between the grains for mortars consisting of 1 part of cement to 2½ parts of sand or richer. For leaner mortars, consisting of 1 part of cement to 4 or 5 parts of sand, such as were experimented with in the French tests by M. Feret, which have been referred to, a mixture of fine or medium sand is desirable to help out the cement. Such mortars, however, are rarely used in American practice. M. Feret's tests indicated that the best sand was a mixture of coarse and fine sands, without any medium sand. The experiments referred to by the speaker indicated that medium sand mixed with coarse sand is more desirable than fine sand.

E. G. HAINES, ASSOC. M. AM. SOC. C. E. (by letter).—This subject reminds the writer forcibly of a remark made to him several years ago, with some heat, by a contractor, to the effect that it was "a mighty easy thing to sit at a mahogany desk and write specifications for sand, but quite another matter to provide it on the work." As a result of that remark, the writer made an examination of the sands found in some 25 sq. miles of country, and came to the conclusion that the contractor was right. Mr. Haines.

There are often sections of country, of quite large extent, in which work is required to be done, and in which no pit sand can be found fit for use in its natural state on account of the large quantity of other material present. On one piece of railroad construction, in West Virginia, of which the writer had charge of eight miles, it was necessary to obtain the sand from the bottoms of the streams. All such material had to be washed, to free it from mud and clay, and in order to obtain the necessary material, every bar in more than 8 miles of the Monongahela River and Ten-Mile Creek was explored, and work often had to be suspended on account of high water. To have obtained the sand elsewhere would have required hauling it a number of miles by rail, and several miles by wagons, over roads which, at certain seasons, were almost impassable.

With the exception of a limited territory near the coast and the mouths of the larger rivers or their branches, the problem is often, not to obtain a perfect sand, but any at all, worthy of the name, within a cost consistent with economical construction; and it is a fact that in and near New York City much sand is being rejected and condemned, which would be hailed with delight by both contractors and engineers on many important pieces of work outside of that territory. The reason for this seems to be, that all the sand used near New York City, is compared with, and expected to be equal to, that known as "Cow Bay" sand, from the immense sand plants on the north shore of Long Island.

This sand occurs as a bluff, from 50 to 150 ft. high, throughout nearly the entire length of the north shore, and, except for a few feet of loam and top soil, constitutes nearly all of Long Island. In the case of the north shore, near the bluff, even this small amount of soil is non-existent, as it has been eroded by the winds and moved inland, so that for some distance hardly any soil exists, and the only vegetation consists of scattered clumps of grass and wild vines. The bluff is usually within 100 ft. of the shore line, with a quite steep face and a nearly level beach. The water deepens rapidly to a depth sufficient to float small vessels and barges, and, where no piers have been provided, small vessels have often been loaded by "laying on" the beach just after high water, and "hauling off" at the next high tide, the vessel being loaded in the interim by wheel-barrows or wagons.

Mr. Haines. The sand itself is nearly pure quartz, a light yellow, and very uniform in color, and would be accepted almost anywhere on its general appearance alone. It is not to be wondered at, therefore, that large sand and gravel plants have been established at which these materials are screened and shipped, principally by boat, for work near the tidal waters about New York City; or, that it is held up as a standard of comparison with other sands.

The familiarity of engineers in this section with Cow Bay sand, however, has largely obscured the fact that it is purely a local commodity, obtainable, except at great cost, in only a limited section; and this has tended to raise the standard of requirements to a point not consistent with economy. It is not uncommon to find in specifications the requirement that sand "shall not contain more than 2% by volume, of loam (silt), when tested by shaking in a closed vessel containing water, and allowing same to settle in the vessel;" or, "measured after pouring off the water containing the loam, and allowing same to settle." Even Cow Bay sand will hardly fulfill this requirement, and the writer has generally found the percentage to run much nearer 5% than 2% with this sand. It has also caused the rejection of much excellent material, obtainable at less cost, which would have usually answered every requirement of the work.

To attempt to enforce such a requirement is a genuine hardship in many sections, and answers no useful purpose. Nevertheless, the attempt is often made, but usually it is given up before the end of the work. Most banks of pit sand or gravel are overlaid with a few feet of fine material, differing in no way from that below except in the fact that it is much finer; and to remove the same before removing the sand is both costly and of no advantage, as it will usually make a better graded material if it is allowed to mix freely in the excavation before screening out the gravel, particularly if the material is stratified in the bank, as is often the case, and provided the amount of fine material is not too great.

It is generally possible to find deposits of sand within a reasonable distance of most works of construction, but they are quite often overlaid with silt or soil to such a depth, that, if the bank be small, or the quantity required be small, the stripping of the top is necessary in order to avoid an excess of the fine material; and an inferior sand is often brought from a distance, in order to avoid delay and the cost of stripping the bank. Under these circumstances, the writer has always been unable to understand why more artificial sand is not prepared and used, as he has often seen excellent material for it close at hand, while inferior natural sand was being hauled from some distance for use on the work. It would seem that a small portable pulverizer, or set of rolls, might be used to advantage, especially where a crusher is in use on the work. There seems to be a deep-rooted pre-

judice against the use of stone dust for sand, and on one piece of work with which the writer was connected, at least 1 000 cu. yd. of crusher dust from trap rock boulders was allowed to go to waste, and natural sand bought, although tests gave results for the dust equal to or better than the sand. Mr. Haines.

The writer has been engaged in engineering construction for more than eighteen years, and during that time he has had occasion to pass judgment on a great many thousand yards of sand, but, while he has made many tests by water and screens, most of the sand has been accepted or rejected on the old-fashioned tests of inspection and rubbing in the hand; and he does not hestitate to say that these are the best tests of fitness which are known to-day. No two banks, no two shipments from the sand bank, and no two loads from the same shipment of natural sand are precisely alike; and, from the nature of the material, it cannot be expected. As a rule, a bank of sand cannot be passed or accepted as a whole, for there are nearly always some portions of it unfit for use, and the best that can be done is to accept or reject the sand as delivered. For obvious reasons, this requires that it be done, generally, while it is still on the wagon or car. A decision, therefore, must be given at once, and this can be done, because any great amount of silt or top soil is readily apparent to the eye, and whether it is "sharp," silicious sand, whether it contains a large quantity of small shale pebbles or whether it is coated with clay, is equally apparent on rubbing it between the fingers or palms of the hands.

The term "sharp" as here used does not refer to the angularity of the grains, and, while the term has been copied blindly for forty years or more, it was not originally understood in that way, nor is it now among mechanics using the sand. It refers to the peculiar property of sound produced when particles of quartz are rubbed against each other. A coated sand, or one containing much shale or other material, fails utterly under this test, and there is as much difference in the sound as between two different tones from a musical instrument. There is really little reason why sand should be angular, as the greatest volume for the surface to be coated, would be obtained if each grain were a perfect sphere. Engineers might just as well require cubical gravel. For years the same arguments against round sand were advanced against the use of gravel for concrete, but, for various reasons, it is now generally preferred to crushed stone. The very action which has produced sand has also tended to make it spherical, and, as a rule, the most angular sand contains also the most dirt or clay.

There is one thing which cannot be determined by the rough tests already mentioned, and that is the amount of vegetable or organic matter contained in the silt, or in the sand itself. However, this can

Mr. Haines. be determined once for all for any bank by taking samples over the area at different points. It is properly a laboratory test, however, and should be made before the use of any sand from the pit is allowed. The amount of organic matter found by Mr. Thompson in the sand which caused failure appears to have been remarkably small to have produced such results; the writer has used sand which he knew to contain a much greater amount without any retardant or weakening action being apparent. He noticed such an action once, however, when, for mixing mortar, he was obliged to use water from a small marsh where the water was stagnant and had a decided color and an oily appearance on the surface; he concluded that it was due to some oil or acid from the decaying vegetation. It is possible that some similar action took place in the case mentioned by Mr. Thompson. It might also be added that the specifications of the Board of Water Supply of New York City, for the great Catskill Aqueduct, allow in sand "not more than 3% by weight of vegetable matter," and, while the amount seems rather large, it is probably justified on account of the character of the country from which some of the sand must be obtained. While it is doubtful if the above amount of vegetable matter has ever been reached on this work, some has been apparent, but no injurious action has been noticed by the writer.

With concrete laid in the usual manner, with a considerable excess of water, a sand which contains much more silt than would be well to allow in sand for mortar may be safely used, as the agitation of the water, from depositing the concrete, keeps much of the finer silt in suspension, and it will pass off over the top of the forms if provision is made for it. If this is not done, it is deposited as a slimy mass on the top of the concrete, sets very slowly, if at all, and should be removed and thoroughly cleaned off before resuming work. For the same reason, it is practically useless to add finely-powdered dry clay to concrete in order to increase its density, and it is also a mistake to carry the grinding of cement to too fine a point, as a large quantity of it will be carried in suspension and deposited on the top of the work, wherever work is suspended.

Mr. Lesley has called attention to the fact that while cement is tested in many ways, sand is used in a proportion of three to one, often without test. This very difference in the volume, as well as in the character of the materials, accounts for the fact. A sufficient quantity of cement must, of necessity, be provided to cover quite a period of time, and, from its nature, it must be stored under cover; while, in the case of sand, it is often difficult to store any large amount. In fact, on large works in New York City, like the Rapid Transit Subway, or the Atlantic Avenue Improvement of the Long Island Railroad, in Brooklyn, of which the writer was in charge of sections, where materials have to be stored in the streets, it is often

difficult to store even a day's supply, and teams are kept constantly hauling. Even in open country, on the Catskill Aqueduct, the writer has often had a large number of teams hauling to a single shaft, and that from different pits, in order to provide for the current day's work. For this reason, any extensive tests are impossible, and the best that can be done is to take test samples from the pit as a whole, and then see that the delivery conforms as closely as possible to the standard adopted. Test samples should be taken at times from the material delivered, as well as from the mortar or concrete used; but, as the material will probably be in the work before the result of any extensive tests can be known, their main value is as corroborative evidence as to the quality of the materials actually used. Mr. Haines.

Cement is a manufactured product, and can and should conform to fixed requirements, while sand is a natural resource, and must be accepted, rejected, or modified to suit requirements. The latter can often be nicely done by varying the amount of cement, and for that reason the method adopted by the Board of Water Supply of New York City, namely, of paying for all cement used in permanent construction at a certain price bid by the contractors, is to be commended. The writer would also recommend that, in the case of large works, the sand and gravel rights be acquired by the companies for whom the work is done, and the product furnished to the contractors at a price just sufficient to cover the money invested, at a reasonable rate of interest.

The writer believes in tests, and he also believes in specifications; but, while a few years ago the latter were so general in character as to be of little use, they are now often so voluminous as to be cumbersome, and consequently little read. Also, at times, they appear to have been written by idealists, without any regard to the natural resources of the country from which the supplies must be obtained.

As "David Harum" says: "A reasonable amount of fleas is good for a dog, it keeps him f'm broodin' on bein' a dog." To paraphrase, the writer would say that a reasonable amount of impurities in sand is good for the sand: It keeps it from being pure quartz, which, in the United States at least, is very hard to obtain.

SANFORD E. THOMPSON, M. AM. SOC. C. E. (by letter).—The discussion has brought out a number of the very interesting points which one encounters in the study of sand for mortar and concrete. The difficulty of judging the quality of the sand by the eye, and the consequent necessity for actual tests, is emphasized by Mr. Humphrey in his comparison of two sands, the tests of which proved the sand apparently poor to be the better. The comparatively small benefit from sharpness of grain, a point insisted upon by the writer for several years, is coming to general recognition. Mr. Thompson.

In connection with Mr. Mills' statement with reference to the tests

Mr. Thompson. of the sand from the Philadelphia Subway, it may be added that while the total sand contained 0.4% of organic matter, the percentage of organic matter in the silt washed from the sand was 6%, or less than the 10% proposed as a maximum allowable limit in the opening discussion. As there suggested, it appears that both the percentage of organic matter in the total sand and the percentage in the silt must be considered. Of course, it must always be borne in mind that the percentages of organic and vegetable matter referred to are based on ignition tests, and not on mechanical separation of the sand.

Mr. Mills' question as to the practical advantage of introducing fine material into concrete for the purpose of making it more water-tight is an extremely important one. As he suggests, local conditions must affect the conclusion, but the character of the structure also is a most important consideration. On a small job, such as a tank, the cost of forms, reinforcement, and incidental expense is so large that an increase in the quantity of cement will form but a small percentage of the total cost, and rich proportions can be used economically. On the other hand, on a large job like a dam or a tunnel, the cost of the cement is a very important factor, and the introduction of a small quantity of fine material for the purpose of using leaner proportions may be of great advantage. If the strength is of small consequence, and the proportions of cement can be reduced by the addition of some inexpensive material like fine sand, hydrated lime, or clay (provided it is pulverized so finely that it can be introduced without danger of forming balls in the concrete), so as to make the proportions 1:4:8 instead of 1:2½:5, nearly ½ bbl. of cement will be saved in every cubic yard of concrete.

Mr. Richardson brings out the variations which may be produced by different grading of sand. It is hoped that he will publish shortly the results of the valuable series of tests made during the construction of the Wachusett Dam.

Although the writer agrees with many of the points brought out by Mr. Haines, he must take very decided exception to the view that the best tests of fitness of sand known to-day are "the old-fashioned tests of inspection and rubbing in the hand." One example out of many which might be cited as indicating the incorrectness of this view is described in the opening discussion. This particular sand was inspected and approved, not only by two or three practical builders, but also by two engineers acting independently, both of whom had had years of successful experience as specialists in concrete construction; and yet the 1 to 3 mortar made from this sand had a tensile strength, at 7 days, ranging from zero up to 40 lb. per sq. in. The quality of the mixing water, as Mr. Haines suggests, occasionally affects the hardening of the concrete, but, in the case referred to, the concrete failed to harden on the job when mixed with water from the town water-works, and the mortar also failed to attain any strength in the

laboratory when mixed with Newton City water, and made up with three different brands of cement, while mortar of standard sand made with the same water and the same cement attained normal strength. Mr. Thompson.

From many such experiences as the one cited, it is difficult to avoid the conclusion already indicated, that for important concrete construction, especially for reinforced concrete work, unless from a bank of known good quality which has been tested previously, tests of the sand are as necessary as tests of the cement.

Finally, it may be well to call attention once more to the two qualities which are of the greatest importance in selecting sand, namely, the cleanness and the gradation of grains. From the preceding remarks it is evident that:

(1).—Sand for concrete must not contain vegetable matter or other ingredients which will prevent or delay the setting and hardening of the concrete.

(2).—Sand for concrete must be selected and the proportions of the mixture chosen with a view to obtaining the strength or the watertightness required for the given structure.

Tests such as those described by the writer in the opening discussion indicate that occasionally a sand may be graded properly in size of grains, and yet may be unfit for use because of the injurious matter it contains. On the other hand, a perfectly clean sand may be so fine and so poorly graded that its use is uneconomical or sometimes even dangerous.

Comparative tensile tests of mortar made from the sand in question and of standard sand mortar mixed in the same proportions at the same time—as recommended by the Joint Committee on Concrete and Reinforced Concrete—are of the greatest importance as showing in a general way whether or not the sand is safe to use. The tensile test will not distinguish the cause of any irregularities, nor will it show wherein the quality of the sand may be improved for strength or watertightness.

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FIRE-RESISTANT CONSTRUCTION OF BUILDINGS.

An Informal Discussion at the Annual Convention, July 8th, 1909.

BY MESSRS. HERBERT M. WILSON, R. W. LESLEY, RICHARD L. HUMPHREY, AND S. WHINERY.

Mr. Wilson.

HERBERT M. WILSON, M. AM. SOC. C. E. (by letter).—The inventory of natural resources prepared by President Roosevelt's Conservation Commission has opened new lines of thought and investigation to the Engineering Profession as well as to other departments of the social fabric.

Architects and engineers have realized in a general way the waste of property values involved in building and engineering construction not designed to resist such destructive agencies as fire and earthquake. A cursory examination, however, of the statistical results presented to the Mineral Section of the Conservation Commission shows the character of the waste by differentiation into its components of lumber, masonry, steel, and miscellaneous building material.

As many of the members of this Society well know, the Conservation Commission was divided into four sections, charged with the study of the waste of natural resources, as follows: (1) Water; (2) Forests; (3) Land, and (4) Minerals. The Federal Commission was fortunate in having at its disposal the accumulation of more than a quarter of a century of research by Government Bureaus charged with the investigations of the problems under consideration, foremost among which were the Geological Survey, the Forest Service, and the General Land Office.

The Secretary of the Mineral Section of the Conservation Commission is Dr. Joseph A. Holmes, Expert in Charge of the Technologic Branch of the Geological Survey, and to the writer, as Chief Engineer

of that branch, was assigned the inquiry into the waste of mineral resources involved in building and engineering construction. He is indebted to Dr. Holmes and to the Director of the Geological Survey, Dr. George Otis Smith, for permission to present to this Society some of the results of this investigation. Mr. Wilson.

The Supervising Architect of the Treasury Department has under his control public buildings costing more than $200 000 000, and is spending each year more than $20 000 000 in the construction of new buildings. At his behest, in order that he may design these buildings so as to render them most resistant to fire and earthquake at a minimum expenditure, inquiries are now being conducted in the field by economic geologists skilled in the study of the building stones, gravels, sands, and clay deposits of the country which are most suitable and most cheaply accessible to the three hundred cities scattered throughout the United States in which public buildings are now under construction or projected. The samples are transmitted to the Pittsburg Laboratory of the Geological Survey for analysis and test as to their qualities either as aggregates for concrete, or building stone or brick.

In co-operation with the Underwriters' Laboratory at Chicago, and independently at the Government Testing Laboratory at Pittsburg, investigations are being conducted in reference to the rates of conductivity, and the fire-proofing and fire-resistant properties of various structural materials, including strength under load before and after subjecting to fire. Tests of the crushing strength of large samples of building stones, of beams, columns, and floor-slabs of concrete and reinforced concrete, and of building brick and tile, are likewise being conducted in order to determine which will give the most efficient results in building construction of the highest type. This feature of the inquiry has in view the fact that the Government should maintain as high a standard of efficiency in the construction of public buildings as is commensurate with economic design, since such buildings are intended to be of a permanent and enduring nature. The magnitude and importance of this work are such that the Government cannot afford to take any risk concerning methods of construction or materials to be used. The fact that the Government does not insure its buildings against loss by fire makes it especially necessary that it be provided against such loss by making them fire-proof. It is estimated that if the public buildings of the United States were insured, the cost to the Government would be more than $600 000 annually.

A startling fact, developed in considering the value and fire risks of Government buildings, is that the total cost of fires in 1907 amounted to almost one-half the cost of new building construction in the country for the year. The cost of building construction in forty-nine leading cities of the United States reporting a population of less than 18 000 000, amounted to $661 021 286, and the cost of building construc-

Mr. Wilson. tion for the entire country in the same year is conservatively estimated at more than $1 000 000 000. A preliminary inquiry had shown that the cost of fires to the people was nearly $500 000 000, for the year 1907.

The preliminary estimate that nearly one-half the value of all new buildings constructed in the United States in one year was destroyed by fire, was sufficient warrant for a thorough statistical inquiry, which would develop the facts for the whole United States, in connection with the specific inquiry for the Government regarding fire-proof and earthquake-resistant construction. This inquiry included the value of the property destroyed by fire, the maintenance of fire departments, the payment of insurance premiums less the benefits returned, protective agencies, and the additional cost of city water supplies, etc. It has developed the fact that the total cost of fires in the country in 1907 amounted to more than $456 485 000, or about $1 250 000 daily. This cost was five times as much per capita as in any country of Europe, and was equivalent to a tax on the people exceeding the total value of the gold, silver, copper, and petroleum production. This fire cost was greater than the true value of the real property and improvements in any one of the following States: Maine, West Virginia, North Carolina, North Dakota, South Dakota, Alabama, Louisiana, or Montana. This means that the total destruction of the real property in any one of these States in a year would not represent a loss greater than that caused by fires in the whole United States.

The actual fire losses due to the destruction of buildings and their contents amounted to $215 084 709, or a per capita loss for the United States of $2.51.

These actual fire losses are greater than the true value of the real property in the States of Utah, Delaware, Florida, Idaho, Wyoming, or Nevada. If a storm or earthquake devastated any one of these States completely, there would be a great outcry. Assistance would be rushed to the stricken people; a mighty attempt would be made to find a remedy for the conditions, and the activity would not cease until the remedy had been found. But, with the fires scattered throughout the country and the loss distributed among hundreds of cities and towns, there is no appreciation of the seriousness of the conditions, and very little attempt to better them. In many cities a big fire loss for the year is looked upon complacently, as if it were a natural and unavoidable condition. The significance of these figures lies in the fact that the loss in the cities of the six leading nations of Europe amounted to 33 cents per capita, or about one-eighth of the above. In addition to this waste in wealth and natural resources, 1 449 persons were killed and 5 654 injured in fires. As convincingly set forth, at the recent conservation meeting of the Engineering Societies in New York City, by Mr. Charles Whiting Baker, the buildings consumed, in the year, if placed on lots of 65-ft. frontage, would have lined both sides of a

street extending from New York to Chicago. A person journeying along this street of desolation would have passed in every thousand feet a ruin from which an injured person was taken; at every three-quarters of a mile in this journey he would have encountered the charred remains of a human being; and, when burned out, at the end of a year, the fire would begin all over again on an even longer street. Mr. Wilson.

An analysis of these fires losses, with a view to making the results of the structural materials investigations useful, not only to the Government, as the greatest consumer of such materials, but also to the various States and municipalities, and to the people of the country as a whole, showed that it would be desirable to point out, not only the most efficient methods of construction for each locality, but also how such improved construction might be brought within the cost limits of all the people. In this analysis, the fact stands out prominently that much of the fire waste is due to fires extending beyond the limits of the buildings in which they started. Exact figures as to the losses due to exposure were not obtainable, but the most conservative estimate indicates that at least 27% of the losses resulted from fires extending beyond the building of origin. Thus nearly one-third of these losses are due to the inflammable construction of American buildings, for in Europe, where fire-resistant construction prevails and building regulations are designed and enforced with a view to restricting the progress of a fire, there is practically no loss from exposure fires. And it is even more significant that only $68 000 000 of the loss from fires in the United States was in brick, concrete, and other slow-burning construction, while double that amount, or $148 000 000, was in frame buildings.

The investigation into the actual cost of fires in the United States was quite thorough, and included a complete inquiry into the total cost of water-works systems and the part of the construction and equipment, the annual expense, depreciation, taxes, interest, and maintenance costs chargeable to fire service; also the total cost of fire departments, the annual expense, depreciation, taxes, interest, and maintenance charges. The results showed $145 604 362 as the amount of fire premiums paid to the insurance companies above the losses paid by them; $28 856 235 as the total annual expense of the water-works upkeep chargeable to fire service; $48 940 845 as the total annual expense of fire departments; and $18 000 000 as the total annual cost of fire protection.

The results of this general inquiry into the causes of fires are summarized in Table 1.

A careful examination of Table 1 tells the story to the engineer. In studying it, he should bear in mind that, in the last thirty years, the total fire waste, being only the value of property destroyed, amounted to $4 484 000 000.

TABLE 1.—CAPITAL INVESTED FOR FIRE PROTECTION, AND ANNUAL LOSS AND EXPENSE ON ACCOUNT OF FIRE, IN THE UNITED STATES IN 1907.

			Investment in construction and equipment.	Annual loss and expense.
FIRE LOSS:				
Total fire loss		$215 084 709	.....	$215 084 709
FIRE PROTECTION: INSURANCE:				
Amount of fire premiums paid above amount of losses paid		(a) 145 604 362	.....	145 604 362
COST OF WATER-WORKS, CHARGEABLE TO FIRE SERVICE:				
Total cost water-works (construction and equipment) chargeable to fire service		(b) 245 671 676	$245 671 676	.....
Source of water supply (construction and equipment) chargeable to fire service	$66 482 220	.....	.....	.....
Distributing system (construction and equipment) chargeable to fire service	2 016 927 tons of metal	127 236 668	.....	.....	.....
Hydrants	350 152 hydrants	29 761 400	.....	.....	.....
Separate high-pressure fire service	...	22 191 388	.....	.....	.....
Total annual expense of water-works, chargeable to fire service	.....	.....	28 856 235	.....	28 856 235
Depreciation and taxes, water-works, chargeable to fire service		10 563 881	.....	.....	.....
Interest charge, water-works, chargeable to fire service		(c) 9 826 867	.....	.....	.....
Maintenance, water-works, chargeable to fire service		8 465 487	.....	.....	.....
FIRE DEPARTMENTS:				
Total cost of fire departments (construction and equipment)			107 063 524	107 063 524	.....
Total annual expense of fire departments			48 940 845	.....	48 940 845
Depreciation and taxes—fire departments		4 603 731	.....	.....	.....
Interest charge—fire departments		(d) 4 282 540	.....	.....	.....
Maintenance—fire departments		40 054 574	.....	.....	.....
PRIVATE FIRE PROTECTION:				
Total cost, construction and equipment, private fire extinguishers, automatic sprinklers, etc			50 000 000	50 000 000	.....
Total annual private fire protection		.....	.....	.....	18 000 000
Total fire waste		.....	$402 735 200	$456 486 151

(a). The amount paid by insurance companies on account of fire loss was $114 164 469, and the amount received by them in premiums was $259 768 831.
(b). This is 22% of the total cost of water systems, domestic and fire service combined.
(c). $245 671 676, cost of water-works, chargeable to fire service, capitalized at 4% interest, is equal to an annual charge of $9 826 867.
(d). $107 063 524, cost of fire departments, capitalized at 4% interest, is equal to an annual charge of $4 282 540.

Mr. Wilson.

Mr. Wilson.

The general inquiry included not only the cost of fires in the United States, but also the cost in Europe, including fire waste, excess of premiums over insurance paid, annual expense of city water-works and fire departments chargeable to fire service, and private fire protection in a number of countries. From the results obtained, it is evident that the United States is paying annually a preventable tax of more than $366 000 000, or nearly enough to build a Panama Canal each year. The estimate gives the total cost of fires in the United States, if buildings were as nearly fire-proof as they are in Europe, as $90 000 000.

The figures are set forth in Table 2.

TABLE 2.—ANNUAL LOSS AND OUTLAY IN THE UNITED STATES, ON ACCOUNT OF FIRES; WITH COMPARISON SHOWING PROBABLE ANNUAL LOSS AND EXPENSE, IF BUILDINGS WERE AS NEARLY FIRE-PROOF AS IN EUROPE.

	United States, 1907.	Per capita.	United States, if buildings were as nearly fire-proof as in Europe.	Per capita.
Total loss by fire	$215 084 709	.....	$41 000 000	.....
Excess of premiums over insurance paid	145 604 362	.....	28 000 000	.....
Annual expense of water-works, chargeable to fire service	28 856 235	.....	6 000 000	.....
Annual expense of fire departments	48 940 845	.....	10 000 000	.....
" " " private fire protection	18 000 000	.....	5 000 000	.....
Total fire waste	$456 486 151	$5.34	$90 000 000	$1.05
Total loss by fire	$215 084 709	$2.54	$41 000 000	$0.48
Annual expense of fire protection	241 401 442	2.82	49 000 000	0.57

In Table 2, and more particularly in the following discussion, it should be borne in mind that, while fires will continue to occur in buildings and destroy them and their contents to a greater or less extent, the fire which extends beyond the building of origin and to its neighbor, and thereby produces a conflagration, does so only because the building construction is of the most inflammable nature. The costs which are here recorded, and exceed in amount those due to fire waste, are preventable only in so far as they are an incident to conflagration conditions as distinguished from fires restricted to the building of origin. The statements set forth relative to the possible reductions in the cost of fire departments, city water supply, and private fire protection charges, must be considered only in connection with those service charges which are due to combatting ever-present conflagration conditions.

The insurance and property loss in the destruction of the City of San Francisco by fire and earthquake in 1906 amounted to about

Mr. Wilson.

$315 000 000. The area burned was 3 000 acres, embracing 520 city blocks, on which 25 000 buildings were destroyed, and it is notable that of these only 3 000 were of brick or stone, and 22 000 were of frame construction.

Since 1866, the conflagrations of the United States have cost, in fire waste alone, $936 551 135. This loss, according to European standards, is needless, and is preventable. If, as it has been shown, the actual cost of fire protection in all its phases is more than the fire waste, it is fair to assume that more than $1 000 000 000 has been expended in this period in an attempt to take care of these conflagrations. This, also, is an unnecessary expenditure, if European standards are considered, and makes a total tax from conflagrations alone since 1866 of more than $2 000 000 000. This is true because the conflagration loss in any of the European countries is almost nothing. It is indeed rare when a fire extends beyond the building in which it starts.

Notable conflagrations in the United States have been those of Chicago, in 1871; Boston, in 1872; Baltimore, in 1904; and San Francisco, in 1906. The year 1908 has a conflagration record exceeding that of 1907, one such disaster alone, that of Chelsea, Mass., involving an insurance loss, not taking into account the incidental losses, of $8 846 879.

The analysis of the fire losses in the United States in 1907 is given in Table 3.

TABLE 3.—FIRE LOSSES IN THE UNITED STATES FOR 1907.

Statistics Gathered by the United States Geological Survey.

	Total.	Urban.	Rural.
Total fire loss	$215 084 709	$107 093 283	$107 991 426
Fire loss: buildings	109 156 894	50 173 625	58 983 269
Fire loss: contents	105 927 815	56 919 658	49 008 157
Total, brick, etc., fire loss	68 425 267	48 908 744	19 516 523
Brick, etc.: fire loss, buildings	31 092 687	19 816 474	11 276 213
Brick, etc.: fire loss, contents	37 332 580	29 092 270	8 240 310
Total frame fire loss	146 659 442	58 184 539	88 474 903
Frame fire loss: buildings	78 064 207	30 357 151	47 707 056
Frame fire loss: contents	68 595 235	27 827 388	40 767 847
Total number of fires	165 257	105 406	59 851
Number of fires in brick, etc., buildings	36 140	25 297	10 843
Number of fires in frame buildings	129 117	80 109	49 008
Loss per capita	2.51	2.54	2.49

From Table 3 it appears that the loss is rather evenly divided between urban and rural population. The large losses in cities and villages are not surprising, for not only do they contain many inflammable buildings filled with valuable property, but their proximity renders them particularly subject to exposure risks. The equally large

losses in rural districts, where buildings are widely separated, can only be explained by the assumption that the remarkable efficiency of the city fire departments prevents even greater losses in urban fires, and that the buildings and contents in the rural districts are generally totally destroyed when fire breaks out. This is indicated in the total of the building losses, which amounts to more than $50 000 000 in cities, while in rural districts the loss in buildings, as distinguished from contents, is nearly $60 000 000. The contents loss in cities amounts to nearly $57 000 000, as against $49 000 000 in rural districts, though the contents of city buildings amount in value to several times those in rural districts. The much heavier losses in masonry buildings in cities as compared with those in the country are due to the rarity of buildings of this character in rural districts. The losses in frame buildings in cities and villages, amounting to more than $30 000 000, as compared with losses in buildings of the same character in the country, amounting to nearly $48 000 000, can be attributed to the efficiency of the fire departments and the utter lack of such protection in rural districts. In European cities, the construction of frame buildings is absolutely prohibited in all municipalities, and few are erected in rural districts, owing to the relatively high price of timber, In the United States, the conditions are the reverse. Until recently timber has been the cheaper, as well as the more easily worked, material, as compared with brick, stone, and steel. More than two-thirds of the total loss in the United States in 1907 was due to frame buildings. The exact losses were: $146 695 442 in frame buildings, and $68 425 267 in masonry and steel buildings. Mr. Wilson.

As shown in Table 4, in those States in which the timber supply is abundant, there is an increased per capita loss of 59 cents over the per capita loss in the comparatively treeless States.

TABLE 4.—THE PER CAPITA FIRE LOSS FOR 1907 IN ELEVEN STATES WHERE TIMBER IS SCARCE, AND IN ELEVEN STATES WHERE TIMBER IS PLENTIFUL.

Statistics Gathered by U. S. Geological Survey.

	Total population.	Total fire loss.	Per capita.
Group 1, States having scarcity of timbers: Iowa, Ill., Okla., Conn., Del., N. J., S. Dak., R. I., Kans., Nebr., and N. Dak.	16 785 460	$38 606 558	$2.30
Group 2, Timbered States: Wash., La., Tex., Miss., Wis., Ark., Mich., Pa., Minn., Ore., and N. C...	25 569 533	73 895 950	2.89

Studying these fire losses by geographical divisions of States—a division generally used by the Census Bureau—as set forth in Table 5, a remarkable feature is the large per capita loss in the Southern

Mr. Wilson.

States, namely, $3.66, or more than $1.00 in excess of the per capita loss in any other division. The cause lies in the fact that the Southern States are well timbered, and, in addition, suffer from the handicap of inefficient fire protection in the cities and villages.

TABLE 5.—THE PER CAPITA FIRE LOSSES FOR 1907 IN THE UNITED STATES, AS SHOWN BY GEOGRAPHICAL DIVISIONS OF THE STATES.

Statistics Gathered by U. S. Geological Survey.

Geographical Division.	Total population.	Total fire loss.	Fire loss per capita.
North Atlantic: Me., N. H., Vt., Mass., R. I., Conn., N. Y., N. J., and Pa.	23 779 013	$59 447 532	$2.50
South Atlantic: Del., Md., D. C., Va., W. Va., N. C., S. C., Ga., and Fla.	11 574 988	25 349 223	2.19
North Central: Ohio, Ind., Ill., Mich., Wis., Minn., Iowa, Mo., N. Dak., S. Dak., Nebr., and Kans.	29 026 645	68 793 148	2.37
South Central: Ky., Tenn., Ala., Miss., La., Tex., Okla., and Ark.	16 368 558	59 908 922	3.66
Western: Mont., Wyo., Colo., N. Mex., Ariz., Utah, Nev., Wash., Ore., and Cal.	4 783 557	12 676 426	2.65

Statistics gathered by United States consular officers in Europe, at the instance of the National Board of Fire Underwriters, showing fire losses in six of the most prominent of the European countries, give the surprisingly low per capita loss of 33 cents. Had the United States such a per capita loss, instead of one of $2.51 for a total estimated population for 1907 of 85 532 761, the total fire waste would have amounted to only $28 623 290, or a saving of natural resources from fire, alone, to the extent of $186 461 419.

These facts are set forth in Table 6, which shows that the average loss per capita is $0.33, as compared with $2.51 in the United States.

TABLE 6.—FIRE LOSSES IN SIX EUROPEAN COUNTRIES.

Statistics Gathered by The National Board of Fire Underwriters.

Country.	Years.	Annual average fire loss.	Population, 1901.	Loss per capita.
Austria	1898–1902	$7 601 389	26 150 597	$0.29
Denmark	1901	660 924	2 588 919	0.26
France	1900–1904	11 699 275	38 595 500	0.30
Germany	1902	27 655 600	56 367 178	0.49
Italy	1901–1904	4 112 725	32 449 754	0.12
Switzerland	1901–1905	999 364	3 325 023	0.30

Table 7 is a comparison of America's needless waste and Europe's prudence by grouping the per capita losses in cities of approximately the same size in the two countries.

Mr. Wilson.

TABLE 7.—Fire Losses in American and European Cities of the Same Size.

Statistics Gathered by U. S. Geological Survey and Bureau of Manufactures.

European losses for 1904.				United States losses for 1907.			
City.	Population.	Fire loss.	Per capita.	City.	Population.	Fire loss.	Per capita.
Paris, France	2 714 068	$1 266 282	$0.47	Chicago, Ill	2 049 185	$3 937 105	$1.43
Frankfort, Germany	334 500	99 492	0.31	Cincinnati, Ohio	345 230	1 971 217	5.70
St. Petersburg, Russia	1 500 000	2 128 541	1.42	Philadelphia, Pa	1 441 735	2 093 522	1.45
Birmingham, England	550 000	226 506	0.41	Baltimore, Md	553 669	916 603	1.66
Sheffield, England	426 686	75 989	0.18	Cleveland, Ohio	460 000	515 194	1.12
Toulon, France	101 602	55 391	0.55	Atlanta, Ga	104 984	225 237	2.15
Bremen, Germany	203 847	78 372	0.38	St. Paul, Minn	204 000	522 447	2.56
Molenbeek, Belgium	63 678	106 150	1.67	Evansville, Ind	63 957	196 702	3.08
Laiken, Belgium	31 121	22 349	0.72	Oshkosh, Wis	31 033	80 500	2.59
Etterbeek, Belgium	23 992	19 504	0.81	Easton, Pa	25 238	32 073	1.27

Mr. Wilson.

Realizing that the loss from fires due to the destruction of property involves only a fraction of the total loss, a careful statistical inquiry was made into the additional cost of water supplies necessary for combating conflagrations. The inquiry developed the fact that in 1907 the water-works systems of the United States were valued at $1 129 247 532, the cost chargeable to domestic service being accepted as $883 575 856, and that chargeable to fire service $245 671 676, or 22% of the whole.

In these systems the pipe used amounted to 7 097 800 tons, of which 5 080 873 tons were used for domestic service and 2 016 927 tons for fire service. The total number of hydrants was 420 394, and the number of hydrants in fire service, 350 152. The cost of hydrant fire service was $29 761 400, and the value of established or contemplated high-pressure fire systems amounted to $22 191 388.

The facts covering the property values involved in additional water supplies for combating fires were secured after much difficulty, and, as any engineer must know, are neither as complete nor as satisfactory as could be desired. To engineers and superintendents of water-works, 5 700 report blanks were mailed and an extensive personal correspondence was taken up in order to secure data which were not furnished on the first call in such condition as would make them useful in the investigation. Only a small percentage of the replies were complete enough for use in tabulation. Under these circumstances, recourse was had to the total cost of water supplies, as secured by the census for cities having a population of 30 000 and more and owning water-works. Data for cities having a population of less than 30 000 were obtained from the "Spectator's Year-Book." These data were then segregated to show the total cost under five geographical divisions of the United States, and, finally, the total cost in each of these geographical divisions was subdivided in order to show the total cost in cities of four classes of population, as follows: Cities of less than 5 000; between 5 000 and 30 000; between 30 000 and 100 000; and with more than 100 000 inhabitants.

By adopting the same plan of classification in tabulating the reports received from engineers and superintendents of water-works, who gave detailed information, it became possible to apply percentages to the total cost of water-works systems for the corresponding geographic divisions and classifications by sizes of cities, and, from these, the results given were obtained. The geographical divisions are arranged in accordance with the different requirements in the various localities, and are the result of personal inspection and interviews with hydraulic engineers and superintendents of water-works in a number of typical cities throughout the United States. In the prosecution of this inquiry the writer received much valuable advice and assistance from many members of this Society throughout the United States, several

of whom spent long hours in going over their plans for distribution in the cities under their charge or with the operation or design of which they were connected. In this manner, only, has it been possible to arrive at any reasonable approximation as to the additional cost of water supply and water distribution made necessary by having a reserve for fire-fighting under conflagration conditions. Mr. Wilson.

This inquiry was not based on the idea that water-works are considered for two purposes of equal importance, viz., domestic water service and fire-protection service, but on what it would cost for water-works systems for fire protection alone. The governing idea has been to ascertain, in any city having a modern water-works system, what it would have cost, in the matter of storage or pumping capacity, in the extent and dimensions of the distributing system, and in the number of hydrants, should the city design them anew with a view to supplying only domestic needs and such minor service as would permit fire fighting, with a view to confining the fire to the building of origin, with the latter built of fire-resistant materials. In the New England and Middle States much consideration has been given to the subject of increasing water supplies which were not originally planned to meet the enormous growth in population which has taken place in the last two decades. Pumping service and distributing systems in the hearts of such cities have had to be entirely replaced with larger primary supplies, and in some have had to be duplicated. In a few cities a separate high-pressure fire service has been installed. On the other hand, throughout the Southern States, this feature of increasing existing supplies has been given little consideration. In the more modern cities of the Central States, the city service was originally designed in the light of the most modern needs and practice. In the Far Western States, especially in the small places, the requirements of domestic supply, on account of irrigation of gardens and city yards, preponderates to such an extent that practically none of the water supply has been considered as of account for fire protection, the domestic supply being of such magnitude as to take care, not only of ordinary fires, but also of conflagrations. This is less true in cities with populations of more than 30 000. In the humid regions the domestic requirements are so great in cities of 250 000 or more inhabitants, that but a small percentage of the water supply need be considered as of account for fire protection. For these reasons the subdivision was made by population as well as by geographical distribution.

Much careful consideration has been given these matters from time to time in the past especially by J. T. Fanning, M. Am. Soc. C. E. In a paper on "Distribution Mains and Fire Service" before the American Water Works Association in 1892, he considers that domestic consumption for all purposes may be treated uniformly as 66⅔ gal. per capita per day, as compared with the very much greater per capita supply

Mr. Wilson. furnished in the large cities on account of fire service and domestic consumption. For a city of average population, say 100 000, he adopted a rate of flow for fire service of 10 000 000 gal. per day, and for combined domestic and fire service, 16 666 666 gal., or about 60% of the water supply on account of fire service. In a later paper, before the same Association, at its Twenty-sixth Annual Convention, Mr. Fanning furnished some additional information on this subject, in which he found that the portion of the main required for the domestic flow at the times when there is also a fire flow, and for different populations, was as follows: for 5 000 population, 11.24%; for 10 000 population, 19.52%; for 25 000 population, 43.75%; for 50 000 population, 57.14%; and for 100 000 population, 88.89 per cent.

The outcome of the investigation by the Geological Survey indicates that, on the average, for the whole United States, 22% of the total expenditure on behalf of public water supplies, or less than one-fourth of the total cost of water-works systems, is due to the additional supplies necessary for protection against fires of such magnitude as may be propagated beyond the building of origin. The range in this additional cost has been found to be practically from zero up to 60% in certain classes of small modern cities.

It will readily be seen that this great cost of water-works chargeable to fire protection could be obviated to some degree if fire-proof construction were as good in the United States as it is in Europe, and to that extent the additional expense and use of necessary materials are waste.

The losses in natural resources due to fire waste are not the only preventable losses. Those resulting from earthquake are also great in amount and are equally preventable by the adoption of proper design and materials in the construction of public and private edifices. Recently, commenting on the most terrible earthquake disaster of modern times, that at Messina, Italy, the Italian Ambassador, Baron Edmondo Mayor des Planches, stated:

"My Government has many things for which to thank the people of America since the earthquake in Southern Italy besides the great monetary assistance and the sympathy we received. We are grateful for the scientific information we have received and the invaluable hints which have been given us by the United States Geological Survey. This information will be of more lasting benefit to Italy than even the great sums of money which have been sent to our people."

This statement of the Italian Ambassador was based on the information given to his country in a bulletin relating to the San Francisco disaster, the work of Richard L. Humphrey, Frank Soulé, and John S. Sewell, Members, Am. Soc. C. E., for the Structural Materials Division of the Geological Survey.

Possible means of reducing the enormous waste due to fires, conflagrations, and earthquakes, are: Mr. Wilson.

(1).—By tests and investigations to determine the relative fire-resisting properties of building materials, the relative rates of heat conductivity of such materials, and the development of systems of construction which offer the maximum resistance to fire. These tests should have in view the classification of the order of merit of the building materials, and the cheapening of cost of construction by the use of those best suited to the purpose, since cheaper materials are not often used through lack of knowledge of their availability and properties.

(2).—By the dissemination of information regarding the less inflammable materials of construction, their strength and durability, the methods of utilizing them in building construction, and the availability of the most suitable of these materials near the locality in which they are wanted.

(3).—By tests and investigations at the sites of such disasters as those at Messina and San Francisco, and by dissemination of information relative to the non-inflammable building materials, and the best method of designing and assembling them to fit them to withstand such shearing and torsional strains as they may be subjected to by earthquake.

(4).—By the enactment of building codes and the enforcement of the same with the view to securing more fire-resistant and more fireproof construction. In European cities the erection of wooden buildings is prohibited, and the oversight of brick, stone, steel, and cement construction is such as to diminish danger from fire due to defective flues, poor electrical wiring, etc., and to confine the fire to the building in which it originates.

It is evident that something must be done to stop the unnecessary waste of structural materials. Certain of these materials, such as wood and iron, are not inexhaustible by any means, but are even approaching exhaustion. In order to obtain the best use from these materials in the future, they must be used with a less lavish hand. Waste means increased cost in the very near future.

The known supplies of high-grade iron ores in the United States are estimated at 3 840 000 000 tons, and unless the present increasing rate of consumption is curtailed, they cannot last beyond the middle of the present century. There are, in addition, 59 000 000 000 tons, or nearly twenty times the amount of low-grade iron ore, which undoubtedly will be used when the conditions of the market warrant it. To increase the life of these iron-ore supplies, it is evident that the people of the United States must soon turn to concrete-making materials, brick, tile, and other clay products, and building stones, as substitutes for the more perishable timber and the more limited metal

Mr. Wilson. supplies. From the above a study of the causes of waste of structural materials is evidently of prime necessity. The first source of such waste has been shown to be fires. A second source, and one closely related to fire losses, is that due to waste of iron and steel placed underground in city water mains or in pumping plants, on account of fire and conflagration protection. A third source of such waste is that due to improper mining methods, whereby the whole product is not extracted; and an even greater source of waste is that due to improper or improvident methods of preparing the minerals for market both in shipment and in the metallurgical processes. Perhaps the largest waste of structural materials is that due to lack of knowledge of their strength. Engineers and architects adopt working stresses for concrete and metal construction from one-fourth to as high as one-eighth of the supposed working strength of the material. This means that from three to six times the necessary quantities of such materials may be used in structures, and, as a result, present systems of building construction are wasteful.

In order that this waste may be brought to the attention of the public and the law-makers, and in order that the discrepancy in cost between wood and less inflammable materials of construction may be reduced, thus encouraging the use of non-inflammable materials, the investigations on which the above data are based were undertaken. It is believed that through dissemination of information as to the local availability of cement-making materials, of gravel and sand suitable for concrete construction, of clay suitable for brick- and tile-making, and through tests and investigations which will show the most appropriate method of mixing and proportioning these materials, and of designing them with the minimum amount of each material which may suffice its purpose, will the cost of construction be reduced and the use of such materials be encouraged.

Within the past few years marvelous strides have been made in the substitution of iron and steel for wood, due to the investigations of engineers, physicists, and chemists into the properties of these materials and the great amount of attention given to their fabrication by manufacturers and architects. More recently the engineering and technical professions have advanced to a great extent the uses of cement in concrete manufactures, but in a vastly greater period practically nothing has been done toward ascertaining the physical and chemical properties and the better modes of manufacture and use of the products of clay and stone. With these objects in view, the Government, as the largest consumer of such materials, is undertaking such tests and investigations as may develop the most suitable of these less perishable building materials for each particular use and locality. These tests have in view the establishing of the physical properties of these materials, the suggestion of improved methods of manufacture with a view to

economy, improved methods of mining and marketing in order to improve the quality, reducing the quantity and cost, and extending the life of such materials. The investigations include the assembling of information relative to the most fire-resisting and fire-proof forms of construction, the former for the prevention of conflagrations due to secondary or exposure fires and the latter for the prevention of the destruction of the buildings in which the fires originate. The tests also include investigations to prevent the loss of materials by the action of salt water on the Isthmus of Panama and in government and municipal sea-walls and piers, and also to prevent the loss of materials by the action of alkalies in the arid region, by electrolysis in cities, and by other destructive agencies. The average architect, engineer, or contractor does not take chances in the use of unfamiliar materials or designs, but confines himself to those of which he has knowledge as having proven moderately successful. Authoritative investigations, therefore, are in progress, with the idea of establishing the strength and properties of each of the materials under varying conditions in widely separated localities, and of pointing out the most economical material for each use. Mr. Wilson.

Such of these investigations as bear on the construction of buildings to resist fire are divided into three groups. A series of such investigations is being carried on in the Structural Materials Division of the Geological Survey under the immediate supervision of Mr. Richard L. Humphrey, for cement, concrete, and reinforced concrete, and metal; under Mr. A. V. Bleininger for title, brick, and other clay products; and under Mr. E. F. Burchard for building stone. Under the Fuels Division, inquiries are being conducted, in co-operation with the National Board of Fire Underwriters, into the relative fire risk in the storage of such liquid fuels as gasoline, kerosene, and alcohol. In the Mine Accidents Division, an inquiry into the risks in storing and handling explosives is being conducted.

The Structural Materials Investigations already undertaken, and, in part, reported on, include fire tests of forty large panels representative of a section of a wall of a building. These panels were constructed of various building stones, brick, terra cotta, concrete building blocks, reinforced concrete, etc., and were subjected to the temperature to be expected in a conflagration. The concrete building blocks and the reinforced concrete panels were made of various grades of river and slag sands, various gravels, broken stones, cinder, etc. The brick specimens were of common, hydraulic pressed, and sand-lime brick. Careful record was made of the method of proportioning and mixing the mortar and the concrete materials. The panels were subjected to freezing and to hot water before and after the test and to drenching through the nozzle of a fire hose, when the maximum temperature had been reached. An effort was made to obtain the maximum tempera-

Mr. Wilson. tures of conflagration conditions, namely, 1 700° Fahr. Thermocouples were placed in the panels in order to determine the rate of temperature rise and the rate of heat transmission from the face to the rear of the panel. The temperature rise at the rear of the panel, in connection with the thickness of the latter, furnished information regarding the transmission of heat to the outer side of a building wall.

The work of the National Fire Protection Association should be known to every member of this Society. This Association includes as active members, the American Institute of Architects, the American Institute of Electrical Engineers, the American Water Works Association, and a number of other national technical engineering societies, fire underwriters' associations, city fire department associations, etc., and the writer believes that it would be to the advantage of this Society to be a member of that Association, in order that closer co-operation might be had along the lines of work on which both are engaged. The associate membership of that Association and its officers are chosen from that rapidly increasing group of professional men known as fire-protection engineers, many of whom are civil engineers, members of this Society, mechanical engineers, electrical engineers, architects, testing engineers, and the more advanced members and officers of the National Board of Fire Underwriters.

Through its committees and in co-operation with the Underwriters' Laboratories in Chicago, and the structural materials laboratories of the United States Geological Survey, valuable data are being collected and important reports issued on the best methods of fire-proof building construction. These reports are discussed at the annual meetings, and result in the preparation of specifications on such subjects as fire-proof construction, concrete and reinforced concrete in building construction, the proper materials to be used in the construction of gravity sprinkling tanks, the proper design of fire-proof window and door frames, elevator shafts, fire-walls, and other subjects of the kind on which the members of this Society who have to do with building construction should be informed.

The writer has trespassed on the time of the Society in bringing to its attention some of these matters, which may appear to be but distantly related to its functions, because it is his confident belief that only through an awakened and enlightened conscience, which can be most conservatively stimulated and wisely directed by engineers and architects, will the public, the producers, the manufacturers, and the governing bodies, embark on the reforms so sorely needed if building construction is to be placed on a more enduring and economic basis.

Mr. Lesley. R. W. LESLEY, ASSOC. AM. SOC. C. E.—There is one point in the fire risk to which the speaker would like to call attention, that is, the legislation in reference to fires in the United States and in Europe.

In France, where he lived for some time, and was familiar with the

law, the owner is responsible to the adjacent property holder for fire extending beyond his own building; in other words, if an owner chooses to build an inflammable frame structure, he takes the risk of burning down his neighbor's property, and he is liable for the damage, and this responsibility probably has a great deal to do with the forms of construction adopted. This policy has been adopted in many other countries of Europe, the speaker believes, and is a part of the law. Mr. Lesley.

RICHARD L. HUMPHREY, M. AM. SOC. C. E.—All engineers, and particularly members of this Society, who have to deal with engineering construction, should give careful consideration to the facts presented in Mr. Wilson's discussion. The summary of these facts points out clearly the numerous annual losses in the United States, which are largely or entirely preventable, and the dangers of the destruction of property by fire which exist throughout the country. Mr. Humphrey.

The conditions which prevail in the United States are not unusual, and are the natural result in a country which has grown with great rapidity. The congestion of population has created problems which necessitate special legislation in order to correct the evils which have resulted from laxity and insufficient municipal supervision in building construction.

It is only after a city has outgrown the flimsy structures incidental and necessary to the settlement of new country that the congestion of population forces on the inhabitants the necessity for greater safety and provision against the dangers from fire.

The development of the building laws must necessarily be extremely slow, particularly because of the great difficulty in securing legislation which will correct the evils of the building laws caused primarily by the commercial influences which effect such legislation.

We are prone to look to Europe, to study the conditions there, and, in drawing comparisons, to calculate what our losses would be if we secured the same types of construction as obtain in European countries. This, of course, is very helpful, and is one of the means of drawing the attention of cities to proper legislation in a practical way. At the same time, it should be borne in mind that European cities are very old, and the congestion of population in the principal cities has been such as to necessitate, many years ago, such revisions of the building laws as would insure proper safety in those congested centers. In other words, European cities have been many years in obtaining perfection in their building laws. The United States is rapidly approaching, indeed it has approached, in many of its larger cities, such a condition in the congestion of its population as to make it absolutely necessary that steps be taken to prevent types of construction which are so dangerous as to form a constant menace of a serious conflagration. Many of these structures are fire-traps, constructed so flimsily that it is almost criminal to permit their existence.

Mr. Humphrey.

Anyone who was so fortunate as to investigate such great conflagrations as those at Baltimore and San Francisco is overwhelmed at the absolute worthlessness of the types of building construction which are in vogue in this country to-day. It is true that, after a fire has gutted a building, the walls and skeletons are found to be standing, and that the building has passed the conflagration satisfactorily. As a matter of fact, what remains of the structure forms a very small percentage of its value, the greater part of the latter having been lost in the destruction of the trimming and the various appurtenances which go to make up the complete building. In New York City buildings which are labeled fire-proof, and are apparently of reasonable fire-resistive construction, may be found to be surrounded by great areas of structures poorly constructed, with unprotected openings of plain glass which cracks at the first touch of the flame and enables a fire from a neighboring property to spread into the building and destroy its contents. Indeed, the intensity of some of the fires which occur in such buildings is so great as to destroy adjacent buildings which are reasonably fire-proof. Even the heat generated by fire in a building which is reasonably fire-proof is sufficient to destroy the integrity of the structure, because the building does not possess the requisite fire resistance. Structures should be designed so as to offer resistance required for the conditions. It is manifest that a warehouse in which large quantities of inflammable materials are stored must have greater protection against fire than a building which contains only a small quantity of such materials. Even where buildings are of the best fire-resistive type of construction known to-day, they are often destroyed by the conflagration of the fire-traps which surround them. For this reason, the building laws should be retroactive, in that they should compel the removal of all structures which are not of reasonable fire-resistive construction, especially in the congested centers of large cities. In the speaker's opinion, buildings should not be permitted to exist which are known to be fire-traps, and particularly should this be the case where the structure is used as a place of assembly for women and children.

In the matter of fire-resistive construction, it is a regrettable fact that most deplorable conditions exist in schools and places of public assembly, and, within recent years, there have been such terrible catastrophes as to horrify the entire nation, and yet these are only sufficient to stir up public sentiment for a brief period of time. As soon as the interest begins to wane, the public drops back into the same condition of indifference, and can only be aroused by a similar holocaust. If the building laws are to be adequately revised within a reasonable time, it is absolutely essential that the foremost societies of the United States, of which this Society should be a leader, should take an active part in the campaign, in order to provide sufficient pro-

tection against fire and thereby reduce the enormous annual losses. This Society has taken an active part in the work of conserving the nation's resources, and certainly there is not a more fruitful field for carrying on this work than in the attainment of structures of such a character as will offer adequate resistance to fire, thereby reducing to the smallest possible amount the enormous annual destruction of building material. Mr. Humphrey.

S. WHINERY, M. AM. SOC. C. E.—This subject is one to which the speaker has not given much attention, but it seems to him that great importance may be attached to the particular fact referred to by Mr. Lesley, namely, the absence of personal responsibility on the part of the owner. Mr. Whinery.

Doubtless, one reason why people are more careless in the United States than in any other country, is that, because of the very complete and efficient system of fire insurance, the losses through fire are not visited on the persons whose buildings are directly burned, but are practically shared by the whole community; and, while fire insurance is undoubtedly a very great benefit to the country at large, it may be a question whether it is an unalloyed blessing.

If the owner could be made to feel that if he puts up a cheap wooden building—a fire-trap, as it has been called—and that building should be burned down, a large part of the loss would fall on him, he would undoubtedly take more care to make it reasonably fire-proof; but, as long as he knows that he can insure the building for nearly its whole value, and thus compel the community at large to share the loss by fire, he will naturally be careless, and will favor cheap construction.

From this point of view, it might be wise, and it might also result in reducing the aggregate losses by fire, if insurance companies were prohibited from insuring any building for more than one-half of its value.

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TRANSACTIONS

Paper No. 1127

THE SEWER SYSTEM OF SAN FRANCISCO, AND A SOLUTION OF THE STORM-WATER FLOW PROBLEM.*

BY C. E. GRUNSKY, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. W. C. HAMMATT, ROBERT G. DIECK, CHARLES E. GREGORY, E. KUICHLING, KENNETH ALLEN, WALTER N. FRICKSTAD, AND C. E. GRUNSKY.

This paper ought to have been written and presented to the Engineering Profession long ago. It is now offered in order that suitable record may be made of the steps taken by San Francisco to better the primitive conditions which have been maintained there too long, and to call attention to a novel solution of the storm-water flow problem which is an outgrowth of the study of San Francisco conditions.

The writer was a member of the 'Board of Engineers' of 1892-93, generally referred to as the Sewerage Commission; he was one of the two 'Engineers in Charge to devise and provide a sewer system for San Francisco,' of 1893; he was in 1899 the 'Engineer in Charge,' with two associates, of the design of the sewerage system, which, with minor modifications, is now being carried out, and he was for more than 4 years, 1900-04, City Engineer of San Francisco, in which capacity he revised and re-submitted the plans of the sewer system, recast the

* Presented at the meeting of May 19th, 1909.

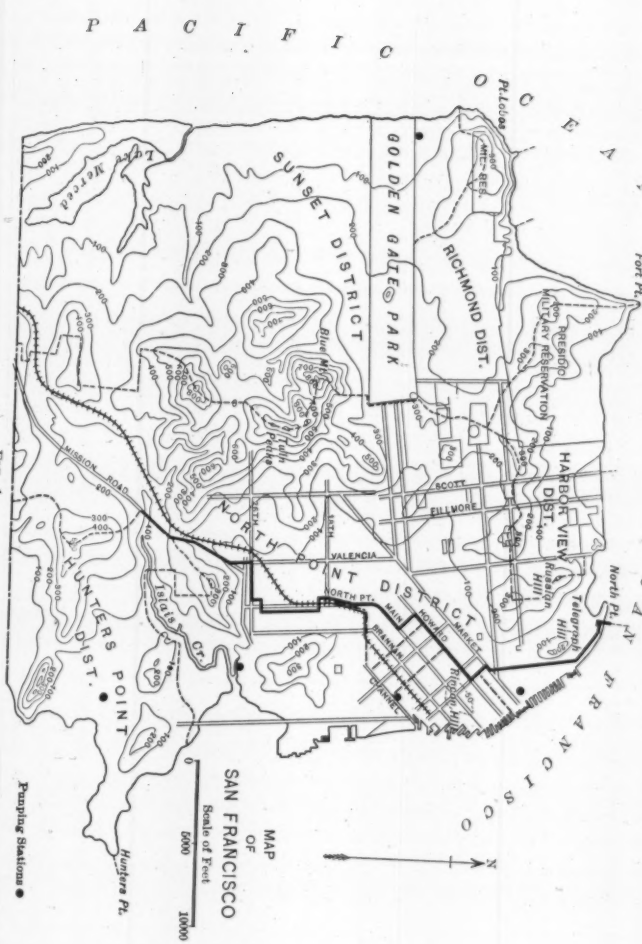

PACIFIC OCEAN
BAY OF SAN FRANCISCO
Pt. Lobos
Golden Gate
Fort Pt.
North Pt.
Telegraph Hill
Russian Hill
Rincon Hill
Lake Merced
SUNSET DISTRICT
GOLDEN GATE PARK
RICHMOND DIST.
PRESIDIO MILITARY RESERVATION
HARBOR VIEW DIST.
NORTH POINT DISTRICT
HUNTERS POINT DIST.
Blue Mt.
Twin Peaks
Islais Cr.
MISSION ROAD
SCOTT
FILLMORE
VALENCIA
NORTH PT.
MAIN
BRANNAN
HOWARD
MARKET
CHANNEL
MAP OF SAN FRANCISCO
Scale of Feet
0
5000
10000
N
Hunters Pt.
Pumping Stations

FIG. 1.

cost estimate thereof, and, as funds permitted, directed the construction of the first elements of the new system. This statement is offered in explanation of the free use which has been made in this paper of the reports on the sewerage problem of San Francisco, prepared or participated in by the writer from time to time.

Topographic Features.

San Francisco covers the entire northerly portion of a peninsula in which a spur of the Coast Range, coming up from the south between San Francisco Bay and the Pacific, terminates. The city is irregularly square in outline. On the west is the Pacific Ocean; on the north, the ocean, the Golden Gate, and the bay; on the east, the bay; and on the south, Marin County, from which San Francisco is separated by an east and west line extending from the bay to the ocean.

The area of the city is about 30 000 acres, of which about one-third is built-up territory. The built-up section covers the northeastern quadrant of the city, and extends, in long narrow strips, westward, in the northern half of the city, to the ocean, and southerly along such principal lines of travel as the San Bruno and Mission Roads to the County line. Within recent years the growth of the section on the ocean slope, particularly the region just south of Golden Gate Park, has been rapid.

The highest point in the city is practically in the city's geographical center. Here are the Twin Peaks which rise to an altitude of a little more than 900 ft. A short distance to the northwest of Twin Peaks is Blue Mountain, with a summit at practically the same height.

The ridge which divides the city into two slopes, one easterly toward the bay, the other westerly toward the ocean, enters the city from the south, a little west of a median line, and holds a fairly direct northerly course to a termination at Fort Point. The lowest points on this ridge are more than 200 ft. high. Before the recent completion of the bay shore cut-off, by the Southern Pacific Company, its main line of railroad into the city crossed this ridge from east to west just south of San Francisco, and within the city crossed back over it from west to east to reach the east central portion of the city. Parts of Golden Gate Park and the Richmond District lie upon the flat top of this ridge northward from the Twin Peaks group of hills.

The principal spurs within the city, from this peninsular backbone, which subdivide the eastern or bay slope, are, in their order from south to north: Hunter's Point ridge, terminating in a cape which extends far out into the waters of the bay in the southeastern portion of the city (south of this spur are two small valleys opening to the bay, of which the southernmost, Visitacion Valley, lies in part in San Mateo County); Bernal Heights, between two branches of Islais Creek; the Potrero Hills, which are barely connected with the main ridge by a slight swell in the ground; and the northern ridge, terminating in Russian and Telegraph Hills on the northeastern bay front of the city.

The western water-shed or ocean slope is subdivided, by the flat-topped Ocean House ridge and the Point Lobos spur, into three drainage basins, of which the northernmost is drained by Lobos Creek; the second embraces a region of drifting sand dunes extending well up on the western slope of the central group of hills; and the third is the northern part of the water-shed of Merced Lake.

Some of the hills near Point Lobos are more than 300 ft. high; the Fort Point ridge, within the Presidio, has a height of more than 400 ft.; the Presidio Heights, near where the northern ridge leaves the main ridge, have greatest elevations of 300 to 380 ft.; Lafayette Square rises to a height of 380 ft.; Clay Street Hill is 360 ft. high; Russian Hill, 370 ft., and Telegraph Hill, more than 290 ft. The lowest point on the spur which extends northwesterly from Russian Hill and terminates at Black Point is about 90 ft. high. On the Potrero Hills are points nearly 340 ft. high, and on the Hunter's Point ridge several hills rise to more than 250 ft. Railroad Avenue crosses this ridge at an elevation of 60 ft., and San Bruno Road at the same elevation. The spur which extends eastward from the main ridge south of the Hunter's Point ridge near the south line of San Francisco, leaves the main ridge with an elevation of 550 ft. This ridge separates Bay View Valley from Visitacion Valley. It is crossed by the San Bruno Road at an elevation of 140 ft., and terminates with a hill the top of which is 400 ft. high.

Bernal Heights rise to more than 490 ft.; Buena Vista Park is nearly 520, and Lone Mountain nearly 480 ft. high.

Between the several ridges or spurs of hills just mentioned lie valleys or broad flat slopes opening out upon the bay or ocean. The

ocean slope water-shed south of the Point Lobos ridge has no well-defined line of drainage. Irregular sand dumes, formed of sand blown eastward from the ocean beach, form a broad broken slope to the ocean. Some of the drifting sand has been carried over the main peninsula ridge just north of the central group of hills, and, separating into two parts, at Lone Mountain, has found lodgment in several broad belts extending eastward and southeastward across the peninsula to Mission Bay. In smaller quantity, some sand has also been carried across the peninsula south of Twin Peaks.

Sand has also been blown inland from the bay beach lying on the northern front of the city to the westward of Black Point. The great sand dunes, now only seen in small remnants, that were formerly at and near Lobos Park and some distance up on the slopes to the east and north, had their origin on this beach.

Where sand has thus drifted over portions of the site of the city it covers clays, shales, and serpentines. To some extent it encroached upon the waters of the bay on the eastern waterfront adding body to the marginal strips of swamp land and mud. Where the natural surface of the city is not covered by sand a clay soil predominates. On the hills a shaly or serpentine rock is frequently cut into when trenching for sewers, gas, or water pipes.

The northern and eastern shore of the city in its original condition was deeply indented by arms of the bay. These have in part been filled in. The most notable of these filled-in areas is the former Yerba Buena Bay, which gave the original name to San Francisco. This extended as far west as Montgomery Street and from the base of Telegraph Hill at the north to the Rincon Hill region on the south. Mission Bay, too, has been cut off and filled in. The extent of this filling will be appreciated when it is stated that here, over an area of about 150 acres, all official street grades are at City Base (6.7 ft. above ordinary high tide).

Islais Creek, south of the Potrero Hills, opens to the bay through broad marsh lands and mud flats, 500 acres of which are within the city limits. It is in this region that, fully two miles inland, official street grades as low as 1 ft. below City Base may be noted.

South of Hunter's Point an area of from 700 to 800 acres within the city limits extends out into the bay over submerged mud flats which will probably some time be filled in to about City Base.

On the northern front of the city the tidal area and bay surface that have been or that ultimately will be cut off from the bay by the established seawall line aggregate about 270 acres.

The total area of originally salt-marsh lands and bay surface within the boundaries of San Francisco is nearly 2 000 acres.

The built-up or improved sections of San Francisco fall naturally into districts of two characters: one class embracing portions of the city on sloping ground, hillsides, and hill-tops, at such elevations that they can be readily drained by gravity flow; the other embracing the low flat areas for which it is difficult or impossible to provide satisfactory sewage collection without recourse to pumping. The several districts of each class, as the foregoing description of the topography indicates, do not lie contiguous to each other. The drainage and sewerage problem is thus rendered much more complex than if there were only one instead of many drainage basins.

Water-shed Areas.—The extent of the several water-sheds, the run-off waters of which are to be cared for was in 1893 estimated as follows:

Visitacion Valley water-shed	990 acres.
Bay View water-shed	1 220 "
*Islais Creek water-shed	3 900 "
Precita Creek water-shed	1 730 "
Mission Bay water-shed	5 860 "
Yerba Buena Bay water-shed	430 "
North Beach water-shed	320 "
Washerwoman's Bay water-shed	1 030 "
Presidio water-sheds	930 "
Lobos Creek water-shed	2 120 "
Ocean Beach water-sheds	5 650 "
*Laguna de la Merced water-shed	3 350 "

In this list Precita Creek has been given independent rank. It is a tributary of Islais Creek, uniting with the latter at the margin of the salt marsh. Bernal Heights forms a prominent line of division between these two creeks.

Visitacion and Bay View Valleys.—Both Visitacion and Bay View Valleys are still sparsely settled, but the latter at least is a section in immediate need of main sewers.

* Only the area of these water-sheds within San Francisco is here noted.

Islais Creek.—The main branch of Islais Creek heads to the southward of San Francisco. It descends rapidly in its westerly course to the low marsh lands which extend far up into the valley of this creek. The creek before emerging upon these marsh lands lies for some distance in a deep gorge which takes a course through private lands. The rectangular system of laying out streets has been adhered to in some places along this creek, where, without question, there should have been streets following lines of drainage. Where lands are not yet subdivided, it is to be hoped that this matter will receive proper attention.

Islais Creek receives the run-off waters from a portion of the San Bruno Mountain, to the south of San Francisco, from the northeast slope of the Twin Peaks group of hills, and from the entire southern slope of Bernal Heights. The region which it drains, the south central portion of San Francisco, is a rapidly growing section of the city.

By statutory enactment, Islais Creek is declared to be navigable "from Franconia Landing near Bay View turnpike to its outlet into the Bay of San Francisco, and thence easterly along the southerly line of Tulare Street to the city water front on Massachusetts Street, of the width of the channel of said creek." However, Kentucky Street has been carried across the creek on a solid fill and there is still some doubt as to whether the creek, which across the marsh is but a slight depression in the mud's surface, will be opened and kept open for navigation purposes. But at the mouth of the creek a large basin should be dredged to a depth of about 40 ft. to meet in part the needs of San Francisco's growing commerce. It is foreseen that a project for such a basin will ultimately be carried out. All the Islais Creek marsh has been laid out in streets, and official grades have been established. Sometimes this was done in the most absurd fashion. Thus, on San Bruno Road and on Fifteenth Avenue, grades are as low and even lower than on Kentucky Avenue, 1¼ miles nearer the bay.

Precita Creek.—This creek drains a region which lies between the Islais Creek water-shed on the south and the Mission Bay drainage on the north. As a drainway, the creek has been replaced by Army Street sewer. The discharge of sewage is upon or through the marsh into Islais Creek, a short distance east of the eastern spur of Bernal Heights.

Mission Bay Water-shed.—The drainage basin tributary to Mission Bay is of large extent, and is for the most part densely populated. It includes the heart of the city, the entire Mission and the Western Addition. All the sewage and storm-water run-off of this region, except a relatively small portion intercepted by Brannan Street sewer, finds its way into the open waterway of Channel Street. The large Channel Street sewer, and the sewers on Seventh, Sixth, and other streets, discharge into this waterway. The discharge from these sewers represents more than half of the sewage and rain-water of the built-up portion of the city. During half the year there is no rain to dilute the sewage, and it ebbs and flows in Channel Street, creating a filthy nuisance beggaring description. There is no chance for sweeping it out into the bay because there is no circulation of water. It will continue to be offensive until the sewage is carried to some other point of outfall. But even under the contemplated arrangement to be hereinafter described the remedy will not be complete. There will still be some, though very dilute, sewage occasionally discharged into Channel Street, and this open water will remain under a disadvantage common to all long narrow inlets from a tidal bay, that of infrequent change of its water body.

The principal sewer of the Mission Bay water-shed, as now in service, is the Channel Street sewer, 11 ft. wide and 8½ ft. from invert to roof. Second in importance, if judged by size, is the Brannan Street sewer, 9 ft. wide and 7 ft. high in the clear. It intersects the sewers of crossing streets; and, by reason of a connection at its head, near Tenth Street, with the Channel Street sewer, is supposed to act in a measure as a relief outlet for it. The grade of the Brannan Street sewer does not conform closely to the grades of the sewers which it intersects. These intersected sewers are generally from 1 to 3 ft. higher than the bottom of the Brannan Street sewer, which thus should become a recipient of the sewage from the entering sewers. As a matter of fact, it is found that the Brannan Street sewer was constructed so carelessly that, where in tunnel, under the south slope of Rincon Hill, there is an offset of 10 or 15 ft., and its bottom grade for several blocks slopes the wrong way. The accumulation of silt in the sewer obliterates the non-conformity of grade with the intersecting sewers and there is therefore a partition of flow at practically every such sewer crossing.

The Brannan Street sewer discharges into the bay under the First Street wharf.

Other sewers, on Mission, Howard, and Folsom Streets, have independent outfalls, under the wharves or piers of the waterfront.

The sewage from the northern slope of the Potrero Hills is discharged by small sewers upon the Mission Bay mud flats or, reaching the Potrero Street sewer, it is delivered into the bay in the Potrero near the Union Iron Works.

Yerba Buena Bay Water-shed.—The name, Yerba Buena, as applied to any section of San Francisco is now practically lost. Yerba Buena Bay, the old harbor of Yerba Buena, the precursor of San Francisco, has long ago been filled in and built upon. The extreme westerly end of the bay, that is to say, the point of greatest indentation, reached Montgomery Street. The name has been used in the various sewer reports to designate the low, flat portion of San Francisco which lies north of Market Street, near the bay, and the water-shed includes the slopes of the hills to the north and the west which send their drainage through this flat area. As a sewer district, though of small extent, it is of particular interest and importance, first, because about 120 acres thereof are of the filled-in character, second, because it embraces the most important business section of the city. In this old part of San Francisco are to be found the most striking examples of sewer construction, without comprehensive plan. Almost every street and alley has its 3 by 5-ft. sewer (see Fig. 2), and at every street crossing there is the typical sewer intersection. North and south streets with level surfaces, that is, without gradient, are no exception to the rule. The street in which the standard brick sewer cannot be found is a welcome exception. Until about 1890 there was an outlet for each east and west sewer at the water front. Then, however, a number of these sewers were diverted on East Street from their direct courses and a common outfall at Washington Street was provided for Market Street sewer and the sewers as far north as Jackson Street. This interception of the sewage from the west, which includes the intersection by the lower Market Street sewer of all flow in the sewers from Geary Street to Sacramento Street, makes the Washington Street outlet a point at which a large quantity of sewage is discharged. This outlet is at the low-water line, under the wharf.

The sewers of Vallejo and Pacific Streets have been turned into the

Broadway sewer, which thus becomes their common outlet. It, also, discharges under a wharf at the low-water line. It follows, from the conditions indicated by this description, that much slush and filth has found lodgment in the sewers of this flat area; and it may be added that the gradual subsidence of a part of the area, and the construction of sewers, a block or two at a time, which did not conform to the other sewers in grade, sometimes being several feet too high or too low, have greatly augmented the evil.

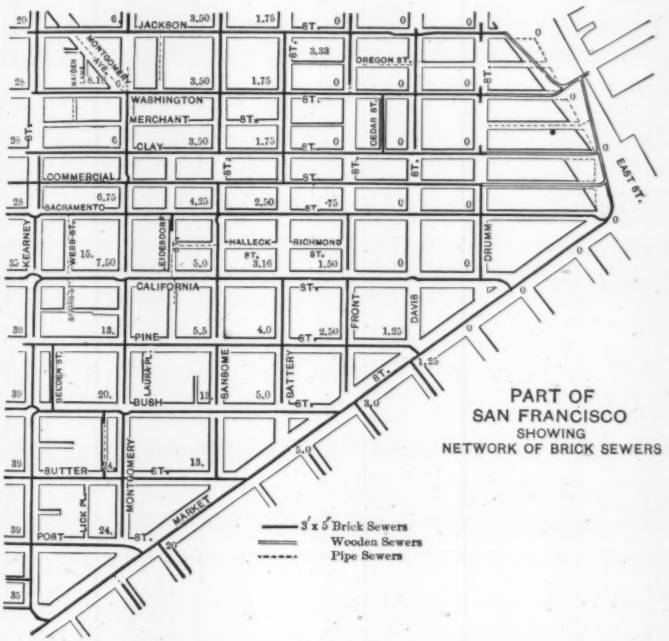


FIG. 2.

The conditions, as they were found when the first examination of this district was made, in 1892, plainly pointed to the use of the existing sewers, in part, at least, as storm-water conduits, and the construction of a new system for sewage only.

North Beach Water-shed.—The North Beach region has a long frontage on the bay. It extends from Telegraph Hill to Black Point.

The main difficulty to be overcome in this region results from the long bay frontage. Sewers now in use discharge into the bay wherever they happen to reach the water line. Sewers of this character are those on Larkin, Hyde, Mason, and Powell Streets.

Washerwoman's Bay Water-shed.—From the Presidio Heights eastward to Russian Hill and Black Point, the northern slope of the northern ridge is toward that part of San Francisco Bay known originally as Washerwoman's Bay. Here, again, the peculiarity of a steep descent from the ridge and the large extent of comparatively flat marginal land is to be noted. Some of the land along the bay remains to be reclaimed.

The built-up area is here fast extending downward on the hillsides toward the bay. Sewers for a long series of years terminated in land-locked basins or at the inshore limit of the tidal marsh. Some of the principal sewers have more recently been extended to the bay shore. The present outfall points of this district are at the bay shore on Pierce Street, on Steiner Street, on Fillmore Street, under a wharf, and on Webster Street.

Lobos Creek.—Lobos Creek had its head originally in Mountain Lake. Though not fed by any well-defined natural lines of surface drainage, it includes within its water-shed all the Richmond District. The southeastern limit of its water-shed is at Lone Mountain. The plateau region northward from the Lone Mountain ridge, westward from First Avenue, and southwestward from the Presidio ridge is drained by this creek. For convenience of classification, all the ocean front drainage, from Fort Point to Point Lobos, may be included in the Lobos Creek water-shed.

The principal sewer of this district is the Richmond main. Here, at variance with the rest of San Francisco, an orderly arrangement of the sewers, which are of comparatively recent construction, may be noted. The outfall of the Richmond main is just west of Baker's Beach, at the foot of Twenty-seventh Avenue.

The Ocean Beach Water-shed.—This region fronts on the ocean, and extends from Point Lobos to Laguna de la Merced. It is without natural lines of drainage, except the small gullies on the western slope of the Twin Peaks group of hills which are soon lost in the deep irregular sand dunes which have drifted in from the ocean.

Laguna de la Merced Water-shed.—Lake Merced lies in the south-

western portion of the city, but a few feet above sea level. The lake has a surface area of about 300 acres. Of its total water-shed of 7 500 acres, somewhat more than one-half lies south of the city in San Mateo County.

The lake is still in use as a source of water for San Francisco, being fed by the waters which percolate through the sands covering a large portion of its water-shed. Some 2 700 acres, of which about 2 300 lie within San Francisco, are owned by the Spring Valley Water Company, around the lake, for its protection against pollution.

The surface run-off from the south is intercepted and turned through a tunnel into the ocean. Likewise, the storm-waters of the Ocean View region are brought within reach of the same tunnel by a long flume. The sewage of Ocean View is carried in a line of cast-iron pipe to an outfall into Merced Creek below the lake.

Studies for a Sewer System and its Design.

The need for a comprehensive system of sewers in San Francisco was pointed out, with increasing frequency and emphasis, by various city officials and committees, until in 1892 the Board of Supervisors, after conference with Frank Soulé, M. Am. Soc. C. E., of the University of California, passed the following resolution, dated March 7th, 1892:

"*Resolved*, That Professor George Davidson of the United States Coast and Geodetic Survey, Col. George H. Mendell of the United States Engineers, and Irving M. Scott, Esq., be and are hereby empowered to select and appoint two engineers, and when the same are so appointed that they, with the gentlemen named, shall constitute a Board of Engineers on and from September 1st, 1892, to devise and provide a system of sewerage for this city and county, as proposed in the report of the Committee on Streets, etc., of this Board, made January 25th, 1892."

At a meeting on August 8th, 1892, the first three members of the Board of Engineers then created:

"*Resolved*, That Marsden Manson* and C. E. Grunsky be appointed as the two civil engineers to complete the Board of Engineers, as contemplated by Resolution No. 6612, adopted by the Board of Supervisors, March 7th, 1892; provided, they agree to devote their entire time to this work after being officially notified by Professor Davidson."

It quickly became apparent to the Board of Engineers thus created, that there were no adequate records of the sewers in use. The sewers

* Member, Am. Soc. C. E., and now City Engineer of San Francisco.

had for the most part been constructed piecemeal, a block or a street crossing at a time, and diagrams to illustrate the work had frequently been considered superfluous. In many cases no record had been made of the completed structures. It was necessary, therefore, to make an examination of the sewers by entering manholes and by exploring the larger sewers. In this way dimensions and conditions as to serviceability, if retained in use as a part of a comprehensive system, were ascertained. Several exploring parties were organized for this purpose, and the results of their work were at once mapped on a suitable scale.

The preparation of a complete topographic map was also undertaken and this was subsequently found to be of great value in facilitating the studies.

Attention was at once given by the Board to the selection of suitable points of outfall for the city's sewage. It was assumed at the outset that only one method of disposal—discharge into the bay or ocean—would come under serious consideration. Therefore, plans were made to observe the direction and velocity of the bay currents.

The work of the Board of Engineers had not been far advanced before its independent course gave offense in certain quarters and led to its dismissal (January, 1893). Its books and the data collected in the four months of activity were by the Board of Supervisors turned over to Mr. Marsden Manson who, however, declined an offer to continue the work. A few days later, after repeated conferences with members of the Board of Supervisors, Mr. Manson and the writer were appointed as engineers in charge to devise and provide the sewer system.

The work outlined by the Board of Engineers was continued, and has become the basis for all sewer work in San Francisco since that time. On July 1st, 1893, the existing sewers, 200 miles in length, had been examined and their general condition noted. All sewers through which it had been possible for a man to pass had been examined as to general condition. In some instances, where data could not be obtained otherwise, the sewers were uncovered. The ascertained facts were tabulated for convenient reference, and were platted on sheets of convenient size on a scale of 100 ft. to the inch. Sixteen of these sheets were at that time complete, ten were in progress, and forty more would be required to cover the built-up areas of the city. At this time, the general map of the city, on a scale of 600 ft. to the inch, had been nearly completed.

Float observations at all stages of the tide were made in the bay from a number of points, principally Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. The purpose of these observations was to make sure that the discharge of sewage at the outfall points, as finally selected, would, by reason of ample dilution, become inoffensive and harmless before any of it drifted inshore.

The work of the Engineers in Charge, however, was not carried forward to the completion of a design of a system of sewers. They shared the same fate as the Board of Engineers—summary dismissal in the fall of 1893—and nothing further was done until 1899 when the Board of Supervisors determined to continue the work under the same engineers but with the addition of the City and County Surveyor, the late Charles S. Tilton; this addition being made as a matter of policy. The resolution authorizing the continuance of the work was passed in May, but, owing to some uncertainty regarding its legality, it had to be re-enacted in modified form. This took time, and it was not until late in September that it seemed certain that the funds set apart for the work would become available. In the meanwhile the work was under way and was rapidly pushed to completion before the close of the year.

Before submission to the Board of Supervisors, the report of 1899 on the sewer system was passed upon by Rudolph Hering, M. Am. Soc. C. E., and received his endorsement.

Some Features of the Problem.

The question of determining whether sewage and rain-water should be carried in the same or separate conduits was necessarily one to be determined for each of the many districts into which the city is divided by its topographic features. The method and place of disposal and the sewers already in service were elements requiring to be weighed in this determination. It was also necessary to make a careful study of rainfall records, in order to ascertain the volumes of storm-water flow that would have to be passed through the sewers to the bay or ocean.

Before taking up the discussion of these matters some additional notes relating to the old sewers, to show the general lack of system in their arrangement, may not be out of place.

The statement, that the plan for providing sewers seems to have been to construct egg-shaped brick sewers, 5 ft. high and 3 ft. wide, in

all streets and alleys, where property was valuable and could afford to pay for large sewers, as applying to most of the sewer work that had been done in San Francisco, is substantially true. The size of sewer required was frequently determined by the Superintendent of Streets, who was never a civil engineer, and the prescribed depth of the sewer, as defined by ordinance, was 10 ft. below the official grade of the street. Therefore, sewers are found, as already explained, where they are all but useless, and types of construction have resulted that are certainly unique. These facts were set forth at some length in the Progress Report of 1893 of the 'Engineers in Charge.'

For years the ordinary improvement of street crossings included sewers across the streets in both directions and storm-water inlets at the curbs on the four corners. The sewers in such intersections were built of brick—four wings from a central manhole. The invert, as required by ordinance, was placed 10 ft. below street grade, generally level, or, due to the intelligence of most of the sewer contractors, a few inches low at the down-hill side of the street intersection. The sewers in the intersection might connect with other brick sewers of like size, or with larger sewers, or with small pipe sewers, according to what was prescribed at some other time for the streets leading from the intersection. After a time, there was a modification of this arrangement of sewers at intersections of streets. Where it was known that the property owners of outlying districts could not afford the large brick sewers, pipe sewers were prescribed for the crossings. The "Bobtail" crossing was introduced. This consists of a section of 5 by 3-ft. sewer 16 ft. long placed along the axis of one street with pipe sewers from its ends and with pipe sewers along the axis of the cross street. A final modification for crossings of small sewers consisted in omitting altogether the brick sewer and putting in two practically horizontal pipe sewers along the center lines of both intersecting streets.

There is no excuse for the many useless storm-water inlets that have been constructed on gutter summits. These occur in all parts of the city which had been improved to any extent before 1890. In 1893, in a central portion of the residence district of the city, a count was made of the storm-water inlets located at corners from which there was a down gradient in both directions, into which, therefore, no water can flow. The district covered by the count extended from Taylor Street on the east to Broderick Street on the west, and from Washington

Street on the north to Hayes Street on the south. In it there are 1 080 storm-water inlets, of which 149 are absolutely useless, and many more receive water from so small an area of street surface that they are to be classed as unnecessary. Each of these useless and unnecessary storm-water inlets, or catch-basins, as usually called, cost the property owners about $90.

In San Francisco, as in most of our cities, the improvement of streets and the construction of local sewers is paid for by the property which fronts on the improvement. The work may be done in districts, as was done in the Richmond District, when the main lines of drainage were put in, and in the case of the Army Street sewer; but, ordinarily, only short stretches of sewer were covered by the official order of construction or in advance of such order, under which there would be a public letting of the work to the lowest bidder, the property owners would be prevailed upon to enter into a private contract to do the work. The favored contractor was at times responsible for the size of the sewer, suggesting it and securing official approval; at other times the Superintendent of Streets and Sewers determined the character and size, at any rate, this matter was subject to his approval. Only in the minority of cases was it possible to refer to a general plan covering some section of the city for sewer sizes.

The city, preceding 1900, had a City and County Surveyor, whose principal duties related to lot surveying. This was an elective office. As the law also required that a City Engineer be appointed by the Board of Supervisors, it was customary for the Board of Supervisors to appoint the incumbent of the elective office to the office of City Engineer. For services rendered in the capacity of Surveyor or Engineer fees were collected, and when special work of any kind was required by resolution or order of the Board of Supervisors, the compensation to be made was usually named therein.

Supervision of construction was in the hands of the Superintendent of Streets. The City Engineer staked out line and grade and issued a certificate, showing satisfactory construction or showing by diagram departures from prescribed alignment or grade. He was not held responsible for the acceptance of work, however great the deviation from a required position might be.

This unsatisfactory method of prescribing and carrying out street and sewer work continued until the present city charter went into

effect, in January, 1900. Since that time a Board of Public Works has had charge of street and sewer construction, and under a well-organized Bureau of Engineering, with a salaried City Engineer at its head, all matters relating to sewer design and construction receive competent and proper attention.

A few diagrams were prepared in 1893 by the Engineers in Charge, to illustrate the lack of system, and peculiarities of the treatment of sewerage problems in San Francisco. Of these only one, Fig. 2, showing the almost universal use of 3 by 5 ft. brick sewers, is exhibited herewith.

Rainfall in San Francisco.

When the studies relating to the design of a comprehensive sewer system for San Francisco were commenced in 1892, an inquiry was at once made of the Weather Bureau for data relating to rates of precipitation. All that could then be learned was that there were no rains of an inch or more per hour, and that therefore the Weather Bureau had not undertaken to record the rate of rain. But it was agreed that the matter should at once be taken up, as it was quite as important for San Francisco to have positive evidence that the maximum rain in an hour was about 0.5 in. as for the cities of the Atlantic Slope to know that 2 in. or more per hour may be expected occasionally.

Therefore, in 1899, when the matter was taken up again, enough information was at command to enable satisfactory conclusions relating to maximum rain intensities to be reached. Mr. Thomas Tennent, a retired maker of nautical instruments, who had for nearly 50 years kept a rainfall record, was of assistance in the rainfall study. His records were placed at the writer's disposal* and were freely used. From them and from those of the Signal Service and United States Weather Bureau (commencing March 9th, 1871), it was found that there had been rain in excess of 2 in. per day as shown in Table 1.

In order that the infrequency of the days on which there is rain in excess of 2 in., and their occurrence, unless as a rare exception, only in the months November to April, may be understood, some further notes on rainfall and climate may not be out of place. Very little rain is expected from May 1st to November 1st. There are no violent

* At the writer's suggestion, these records were turned over by Mr. Tennent to the U. S. Weather Bureau. They were lost in the fire of 1906.

thunderstorms, as in the States of the Atlantic Slope, the Middle West and the Central Northwest. Even in the winter, rain storms accompanied by thunder and lightning are of rare occurrence.

TABLE 1.—RAINFALLS, AT SAN FRANCISCO, IN EXCESS OF 2 INCHES PER DAY.

Date.	Inches.	Date.	Inches.	Date.	Inches.	Date.	Inches.
Jan. 29, 1850....	2.20	Jan. 9, 1862....	3.50	Feb. 20, 1867....	2.12	Dec. 23, 1884...	2.01
Dec. 17, 1852....	3.00	Jan. 10, 1862....	2.46	Feb. 21, 1867....	2.22	Nov. 23, 1885...	2.58
Dec. 25, 1852....	2.54	Jan. 16, 1862....	2.46	Dec. 18, 1871....	2.83	Dec. 21, 1885...	2.78*
Jan. 7, 1853....	2.06	Jan. 17, 1862....	2.64	Dec 19, 1871....	3.12	Jan 23, 1886...	2.35
Mar. 27, 1853....	2.85	Feb. 21, 1862....	2.09	Dec. 23, 1871....	2.48	Feb. 4. 1887...	2.22
Apr. 16, 1853....	3.45	Nov. 26, 1864....	3.98	Jan. 8, 1872....	2.36	Feb. 5. 1887...	2.92
Mar. 13, 1854....	2.25	Dec. 12, 1864....	2.56	Nov. 29, 1872....	2.06	Mar. 13, 1889...	2.54
Oct. 21, 1858....	2.06	Jan. 17, 1866....	2.17	Nov. 23, 1874....	3.98	Feb. 15. 1891...	3.38
Mar. 26, 1861....	2.53	Jan. 20, 1866....	2.22	Nov. 17, 1875....	2.30	Dec. 29, 1891...	2.21
Dec. 26, 1861....	2.02	Dec. 19, 1866....	4.28	Mar. 5, 1879...	2.73	Nov. 23, 1896...	2.00
Jan. 5, 1862....	2.67	Dec. 20, 1866....	3.62	Jan. 29, 1881....	4.67	Mar. 22, 1899...	2.15

* In 11 hours.

The cyclonic disturbances that pass over the country from west to east—usually from two to four per month—are generally of great extent. When they are accompanied by rain, there is ordinarily a steady downpour at a moderate rate until the area of low barometer has passed on to the eastward, when the change of wind direction from southeast to southwest and the fall in temperature may be accompanied by heavy clearing-up showers. These showers, which are not frequent, are generally local. They may cover only a small portion of the area of San Francisco, and generally have sharply defined limits. Even when the entire city is shower swept, the heavy fall of rain is not simultaneous in all parts of the city. Neither are the maximum rates of rain long sustained. The fact that the maximum rainfall in one hour is well below 1 in. is clearly established by all the records of rain that have been kept in San Francisco. On this point, under date of June 29th, 1899, Mr. Thomas Tennent says:

"In reply to your inquiry concerning the rainfall in San Francisco, and what has been the greatest quantity which has fallen in a given time, say one hour, I will state that I have personally kept a record of the rainfall in this city from 1849 to 1896, measuring the quantity in each 24 hours at 8 A. M. On looking over my records [preceding the establishment of the U. S. Signal Service in March, 1871] I find that the heaviest rainfalls we have had in 24 hours occurred, viz.: 1864,

November 26th, 3.98 in., with the remark, 'Heavy rains in the morning; rain and gale at midday; steady rain P. M. and evening.' 1866, December 19th, 4.28 in., 'heavy rain and gale in the morning; hail with thunder at midday, rain during the balance of the day and evening.' 1866, December 20th, 3.62 in., 'high wind and cloudy in the morning; then a steady rain all afternoon and evening.' I can state without hesitation, that we have never had a fall of rain of 1 in. in any one hour."

Based on the records available in 1899, it was found that:

The average number of rainy days per year is 66. There are to be expected no days on which it rains 5 in., and the probabilities are:

1 day in 50 years with more than	.....	4.50 in.
1 " " 25 " " " "	.....	4.00 "
1 " " 10 " " " "	.....	3.50 "
1 " " 6 " " " "	.....	3.00 "
2 days " 5 " " " "	.....	2.50 "
1 day per year " " "	.....	2.00 "
2 days " " " " " "	.....	1.50 "
6 " " " " " " "	.....	1.00 "
9 " " " " " " "	.....	0.75 "
16 " " " " " " "	.....	0.50 "
26 " " " " " " "	.....	0.25 "
40 " " " " " less than	.....	0.25 "

The frequency of rain and the number of days on which rain fell in excess of the several amounts above named is shown by the diagram, Fig. 3, as determined by actual count for a period of 50 years, from 1849 to 1899. The close agreement between the fact and the probability, indicated by the curve, is noteworthy.

From this study it was concluded that rain storms with excessive fall of rain are rare; that nearly two-thirds of the rainy days are days with less than ¼ in. of rain; and that any day with rain in excess of 5 in. is highly improbable.

It should be stated, too, that on the days that show light rain the precipitation is usually well distributed, the rain is light and continuous during the greater part of the 24 hours, more frequently than in the form of showers of short duration.

The data secured by the U. S. Weather Bureau subsequent to the inquiry of the Board of Engineers in 1893, relating to rain intensity,

were carefully studied. As already stated, there had been a sufficient number of heavy rains to enable satisfactory conclusions to be reached.

On	January 20th, 1894,	it	rained	0.15	in.	in 5	min.
"	October 12th, 1899,	"	"	0.09	"	" "	"
"	November 23d, 1896,	"	"	0.08	"	" "	"
"	January 20th, 1894,	"	"	0.17	"	" 10	"
"	November 23d, 1896,	"	"	0.14	"	" "	"
"	November 23d, 1896,	"	"	0.55	"	" 1	hour
"	January 20th, 1894,	"	"	0.36	"	" "	"

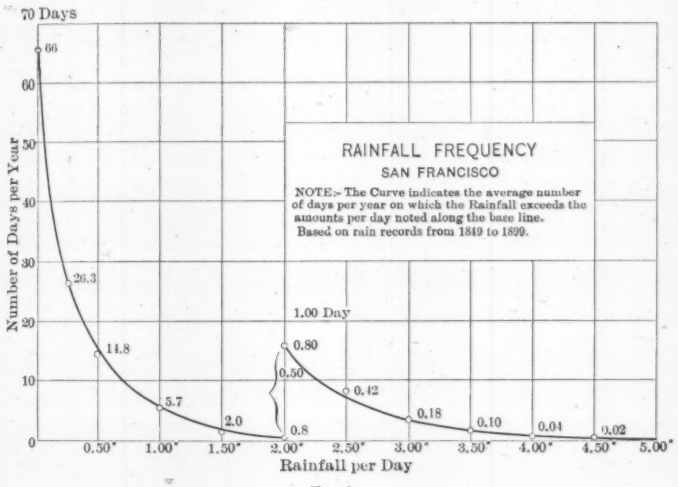


FIG. 3.

Using these and other data from the records, a curve of rainfall intensity was prepared, or rather a limiting curve to show the maximum amount of rain to be expected in any time unit up to 24 hours. The lower portion of this curve was platted on an enlarged scale. This curve is shown by Figs. 4 and 5.

Based on rainfall measurements made subsequent to 1899, a new curve has come into use, which shows somewhat greater rain intensities for short time-periods. The curve now in use is shown in dotted lines.

The most important fact relating to rain intensity disclosed by

the U. S. Weather Bureau records since 1899* is that there was one rain storm in the spring of 1904 which exceeded the intensities indicated by the curve now in use, by about 10% for all periods of time up to one hour. Only two other rain storms were of greater intensity than those indicated by the curve, but the excess in one case was only for a 5-min. period, in the other for a 10-min. period. It is believed that the heavy rain in the spring of 1904 did not cover the entire city.

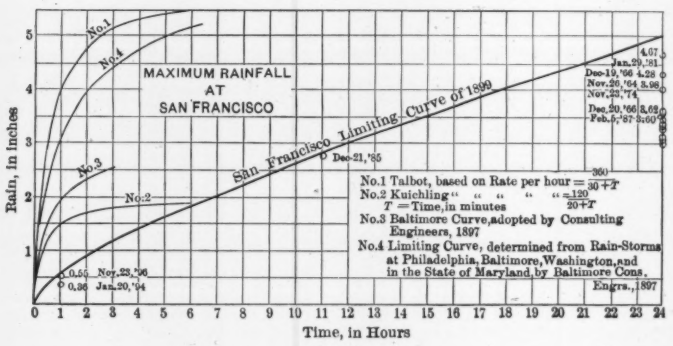


FIG. 4.

The interpretation of the diagram prepared in 1899 established the following for San Francisco:

The maximum amount of rain to be expected:

In	any	5-min.	period	is	0.15 in.
"	"	10	"	"	0.22 "
"	"	20	"	"	0.32 "
"	"	30	"	"	0.40 "
"	"	40	"	"	0.47 "
"	"	50	"	"	0.54 "
"	"	60	"	"	0.60 "
"	"	2-hour	"	"	0.90 "
"	"	3	"	"	1.15 "
"	"	6	"	"	1.83 "
"	"	12	"	"	3.00 "
"	"	24	"	"	5.00 "

* From facts furnished by H. De H. Connick, Assoc. M. Am. Soc. C. E., Assistant City Engineer.

It was confidently assumed that these quantities of rain would rarely be exceeded, and, if exceeded, that the area covered by the excessive rain would probably be small; that therefore these quantities might safely be accepted in estimating maximum storm-water run-off.

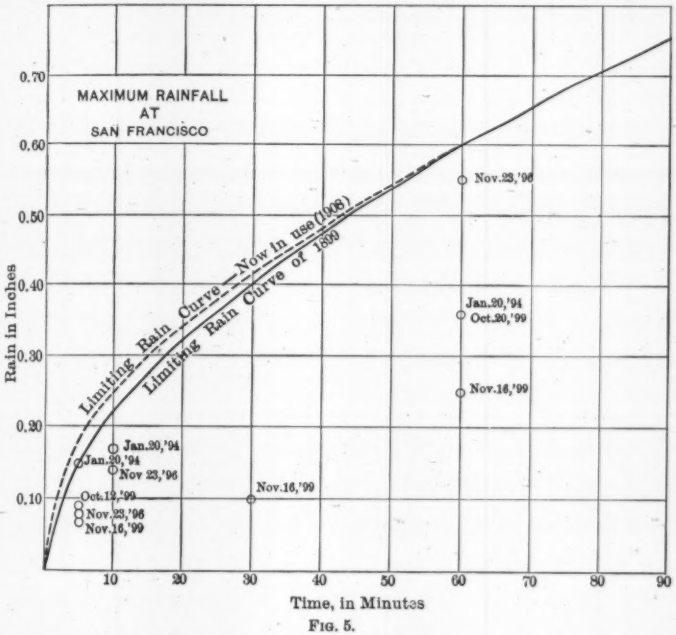


FIG. 5.

Bearing in mind the fact that a rain expressed in inches per hour is practically equivalent to the same number of cubic feet per second per acre, it follows from the above that it may rain in San Francisco as follows:

Duration, in minutes.	Rate, in inches per hour.	Equivalent, in cubic feet per second per acre.
5	1.80	1.80
10	1.32	1.32
20	0.96	0.96
30	0.80	0.80
40	0.70	0.70
50	0.65	0.65
60	0.60	0.60

This is represented graphically in Fig. 6. The ratios above noted were used in estimating the required capacity of storm-water sewers. They have been modified somewhat since 1899, as stated, for the shorter time periods, so that at present, when a sewer for a small district is designed, somewhat larger values for storm-water flow are obtained.

For comparison, the limiting rain curve now in use is shown, as already stated, in Fig. 5, and the corresponding maximum rainfall rates in Fig. 6.

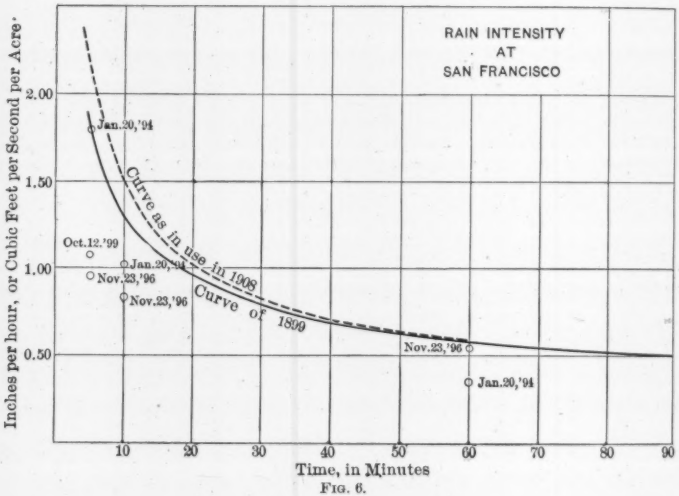


FIG. 6.

Table 2 is the rainfall rate table now in use.

TABLE 2.—RAINFALL INTENSITY AT SAN FRANCISCO, AS IN USE IN 1908.

Time, in minutes.	Intensity, in inches per hour.	Time, in minutes.	Intensity, in inches per hour.
5	2.16	20	1.02
6	1.97	30	0.83
7	1.81	40	0.72
8	1.69	50	0.65
9	1.59	60	0.60
10	1.50	70	0.56
15	1.20	..	....

In order that the great disparity between the maximum rainfall rates to be expected on the Pacific Coast and those of the Atlantic

Slope may become apparent, Fig. 4 shows several other limiting rain curves, in part taken from the report of the Board of Consulting Engineers for the City of Baltimore.* The limiting rain curve adopted by this board for Baltimore, and also a limiting curve determined by rainfall in Washington, D. C., in Philadelphia, Baltimore, and in the State of Maryland, are reproduced in the diagram, and also the limiting rain curves suggested by A. N. Talbot, M. Am. Soc. C. E., and by Emil Kuichling, M. Am. Soc. C. E.

The rainfall intensities indicated by the San Francisco curve of 1899 can be expressed by a formula of the type

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Here I is the greatest possible mean rate of rainfall (the maximum intensity), expressed in inches per hour, when t represents the duration of the downpour, in minutes. The numerator, b, is a constant for any locality, but will vary within wide limits for different localities.

For San Francisco, $b = 3.68$.

For points on the Atlantic Slope, b will generally have a value lying between 8 and 12.

In Table 3 rain intensities are noted for several values of b, and, for comparison, the rain intensities as deduced from the limiting rain curve of 1899 are also noted.

The Estimate of Storm-Water Flow.

Method of 1899.

Having determined the maximum rainfall rates, some method had to be adopted of estimating, from these rates, the maximum rate of flow to be expected at any point of the storm-water conduits. Such method must of course be applicable to areas of all manner of shapes and sizes, of varied surface topography, and of varied character so far as determined by the improvements, present and prospective.

It is obvious that the maximum rate of flow at any point of a storm-water conduit will be less than the maximum rate of rainfall upon the tributary area. Some of the rain will be taken up by porous surfaces, some of it will evaporate, and at the time that the flow is a maximum much of the rain-water will still be scattered over the

* Report of Sewerage Commission, City of Baltimore, 1897.

surface or will be in the conduits the filling of which retards the passage of and elongates the flood wave.

TABLE 3.—MAXIMUM INTENSITY OF RAINFALL, AS DETERMINED IN 1899 FOR SAN FRANCISCO, AND BASED ON THE FORMULA:

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}$$

Duration of rain, in minutes.	San Francisco.			Atlantic Slope.	
	From curve of 1899. Inches per hour.	By formula:		By formula:	
		$b = 3.68$ Inches per hour.	$b = 4$ Inches per hour.	$b = 8$ Inches per hour.	$b = 12$ Inches per hour.
5	1.80	1.80	1.95	3.90	5.85
10	1.30	1.32	1.43	2.86	4.28
15	1.10	1.10	1.19	2.38	3.57
20	0.95	0.97	1.05	2.10	3.15
30	0.80	0.81	0.87	1.75	2.62
40	0.70	0.72	0.77	1.55	2.33
50	0.65	0.65	0.70	1.40	2.11
60	0.60	0.60	0.65	1.30	1.95
90	0.50	0.51	0.55	1.10	1.65
120	0.45	0.45	0.49	0.98	1.47
180	0.38	0.387	0.42	0.84	1.26
240	0.34	0.350	0.38	0.76	1.14
1 440	0.20	0.182	....	....	....

Experience in other cities indicates that the combined effect of these causes, which contribute to make the rate of flow in the conduit less than the rain rate, can with some degree of approximation be expressed as a percentage based on the proportional part of the drainage area which is impervious. This fact has been pointed out by Mr. Emil Kuichling,* and also the further fact that density of population may be used as a guide in determining the correction factor to be used.

It was entirely out of the question to make experiments in San Francisco to be used in the approximation of reduction factors. The assumptions made, therefore, were based on the experience in other cities. In San Francisco, as has been explained, the demarkation of the area tributary to any storm-water main is not possible without first modifying the sewer crossings. But, even if it had been possible to prepare one or more areas for observation, it would have been folly to do

Transactions, Am. Soc. C. E., Vol. XX, p. 1.

this in the hope that some storm might occur within the short time available for preparing the report, particularly as results were wanted before the next wet season. It is to be remembered, too, that such results would have been of slight value, because they would have represented run-off rates for conditions as they prevailed in 1899 and not for those that will prevail in the future.

The study of population distribution was based largely on the records in the Registrar's office. From the size of voting districts and the number of votes in each, with due allowance for the class of people inhabiting each, an estimate of population density was made. Based on this study, it seemed proper to regard all parts of the city having a population of 100 or more per acre as impervious. A population density was then assumed for each sewer district such that the aggregate in all districts would be 1 000 000.

Table 4 shows the reduction factors finally used in estimating the required storm-water capacity from the rate of rainfall.

TABLE 4.

Population per acre.	Reduction factor.	Population per acre.	Reduction factor.
20	0.30	70	0.65
30	0.40	80	0.70
40	0.50	90	0.73
50	0.55	100	0.75
60	0.60	...	....

The procedure in determining the required storm-water capacity for any point of a conduit was substantially as follows: The time was approximated which would elapse before the water falling as rain in the most remote part of the district would reach the point at which the capacity was to be determined. Distance and fall between governing points along drainage lines enabled a quick and sufficiently close approximation of this time to be made. From 3 to 5 min. were then added as the time allowance for the rain-water to collect in gutters and flow to the storm-water inlets. Although, within this elapsed time, there may have been shorter periods with higher rates of rainfall, it is to be assumed that these were practically coincident throughout the district, and that much of the water falling at the maximum rate upon nearby portions of the area had already passed before the water from more remote sections arrived. Consequently, the average rate of

rainfall during the entire critical period was alone taken into account. This was the maximum rate of rain for the estimated time, as shown by the curve in Fig. 6.

The reduction factor, determined by population density, or otherwise, was then applied. In this way the maximum run-off rate per acre was estimated from point to point for each main sewer. Where two main lines of sewers came together, independent estimates were made for each, and the results were combined, if the two districts were not too dissimilar in shape and extent. Otherwise, the time determination, as made for the larger district alone, was used in calculating the rate of run-off. This was done because the maximum flow in the drainway of the smaller district would occur earlier than in the larger district and the allowance for required capacity might become unreasonably large.

It is to be understood that this method of calculating storm-water flow was applied with care. Cases are readily conceivable where the shape of a contributing area is such that it would be improper to apply to the entire area a rain rate determined from the time required by water to flow from the most remote point thereof. The area under consideration may have one or more finger-like projections, long and narrow, which, if taken into account, would make the time long and the rain rate small. In such cases it becomes necessary to examine the lower compact portion of the area by itself. The rain rate is determined in the usual way, and if its application to the partial area—particularly if, owing to surface character, a larger proportion of run-off is to be assumed—gives larger results than the treatment of the district as a whole, then this value determines the required conduit capacity for storm-waters. Sometimes a succession of trials is requisite to establish the maximum possible value.

This method, in its entirety, however, is unsatisfactory, mainly for the reason that it leaves too much latitude to the judgment of the engineer. It has not found general acceptance.

Similar and yet more serious objection may be urged against the use of the various formulas for storm-water flow that have from time to time been suggested by hydraulicians for application to urban areas. None of these, moreover, could have been applied to San Francisco conditions without special study relating to the determination of constants to fit them to local conditions. Even then there would have been

grave doubt as to whether formulas predicated on rainfalls exceeding those at San Francisco from two to six-fold for time periods of one hour and less should be considered applicable. It is only necessary to glance at the rain curves in Fig. 4 to appreciate the force of this statement.

In all the formulas that have come to notice the rate of rain, with some suitable exponent, generally unity, appears as a factor. The rain rate to be used in the formulas must be determined by the engineer, whose judgment is guided by the available rainfall data and his past experience. Thereupon a correction factor, based upon the character of the surface of the district, must be adopted. Even when rain data are ample and the constants have been determined with care the result may be rendered more or less doubtful if the district under consideration has an unusual shape, or has exceptional hypsographic features.

These considerations led to the conclusion, in 1899, that it was not practicable to adapt any of such well-known formulas as those of Hawksley, Adams, McMath, or Bürkli-Ziegler, to the conditions prevailing in San Francisco.

The method already described, appearing in 1899 to be the most rational then known, was therefore adopted.

But the need was felt, and has not heretofore been met, of some more satisfactory method of translating rain rates into storm-water flow. The result of a further study of this subject can now be presented with the hope that it may prove of some value to the Engineering Profession.

New Method of Estimating Required Capacity of Storm-Water Conduits.

Surface Run-off.—The rain which falls upon any area during the time in which the run-off rate is increasing is in part accounted for by infiltration into porous soils and by evaporation; in part it flows past the outfall point of this area, and in part it has swelled the water contents of the storm-water conduits within the area, or is still in transit across the surface to the storm-water inlets.

It has been stated that the method of estimating sewer capacities, as used in the San Francisco problems, takes into account, after a fashion, the retarding influence of temporary water storage on the ground and in the conduits. The new method makes this storage,

which is capable of approximation, an essential element in the calculation of storm-water flow.

This method is applicable wherever it is possible to determine the volumetric increase of the water actually in the conduits during the progress of a rain of the extreme type. The first step, again, is the determination of the maximum rain rates for increasing time intervals. Thereupon, with these rain rates as a guide, the rates can be determined at which the water will run from the surface of the ground. The final step is the modification of these surface run-off rates by the effect of water accumulation or storage on the surface and within all conduits which are located above the point at which storm-water capacity is to be determined.

It is to be remembered that the percentage correction, heretofore noted as applicable to rain rates when storm-water flow is to be estimated by the method used in 1899, covers, not alone the reduction of flow due to water absorption and evaporation, but also the retarding effect due to temporary water storage. These correction factors, therefore, cannot be used when it is desired to estimate for a relatively short time during the progress of a heavy rainfall how much of the water reaches and enters the storm-water inlets.

The greatest intensity of rain is rarely at the beginning of a storm. For the purpose of estimating conduit capacities it must be assumed to occur at a time when the ground is already wet and no longer capable of absorbing water as rapidly as at the beginning of the rain. The rate of water absorption by porous ground plus evaporation, though possibly several times greater at the commencement of a downpour than after the same has continued for some time, by no means keeps pace with the rain rate. It follows that it would not be correct to apply a reduction factor, ascertained from aggregate quantities of rain and of run-off produced by the storm, to the maximum rain rate in order to estimate therefrom the maximum surface run-off rate.

The amount of water which evaporates, during the short time covered by the critical periods involved in urban run-off problems, will be relatively small and, for all practical purposes, may be disregarded.

The run-off, consequently, from an impervious area, the highest supposable type of urban area, during any critical period, measured on the ground where the rain falls, may be regarded as equal to the

maximum rain intensity for that period; and the rate of inflow into storm-water inlets will be this run-off rate, modified only by the effect of changes in the volume of water temporarily stored on the surface of the area.

It will be shown, hereinafter, that the amount of water actually lying upon or flowing over the exposed surfaces of the area under consideration cannot be materially different, at the end of a critical time period, from the amount thus covering the area at the beginning of the period, except when the critical time period is of short duration. It may be neglected whenever the critical time period exceeds 15 min.

The amount of water in temporary storage on the ground's surface, that is, the water momentarily in transit, on roofs, on lawns, in gutters and otherwise, to the storm-water inlets, can be estimated if the time is known that it will take water during heavy rains to flow to the inlets from the outer portions of the small areas served by the individual storm-water inlets. This time is usually from 5 to 10 min. Make the unfavorable assumption that it is 5 min.

The area tributary to each inlet may now be regarded as subdivided into 5 concentric zones, from the innermost of which water will reach the inlet in the average time of $\frac{1}{2}$ min., from the next in $1\frac{1}{2}$ min., and so on to the last one in $4\frac{1}{2}$ min.

At the end of any 5 min., supposing the rain to fall at a uniform rate during this time, there will still be on the ground, flowing toward the inlet, $\frac{4.5}{5}$ in. of the rain which fell on the outer zone, $\frac{3.5}{5}$ in. of the rain which fell on the next smaller zone, and so on down to the innermost zone, on which there will still be one-half of the rain which fell during the last minute. (No correction has been made for the fact that the outer half, by time, of each zone is somewhat larger than the inner half.)

If, now, the area which is served by an inlet be considered as circular, with the inlet in the center of the area, then the areas of the successive zones and the entire area will be to each other as 4, 12, 20, 28, 36, and 100.

Calling the total amount of rain which fell upon the area in 5 min. $100x$, then the amount of water remaining on the ground at the end of this period will be

$$\left(\frac{4}{10}+\frac{36}{10}+\frac{100}{10}+\frac{196}{10}+\frac{324}{10}\right)x = 66x.$$

The water on the ground at the end of the 5-min. period, therefore, is $\frac{66\,x}{100\,x}$, or two-thirds of the total amount of rain which fell in that period.

Generalizing, it may be said that what is true of an area with circular outline is true of other compact shapes, and that, therefore, two-thirds of the water which falls during any time period equal to the time required by water to reach the inlets from the outer portions of areas drained by each, will, at the end of that period, be still on the surface in transit to the inlet.

For any limiting rain curve, therefore, the amount of water on the ground in a district with impervious surface can be estimated for any instant of time, if the time is known that will be consumed by the water in flowing from the limits of an inlet area to the inlet.

For the inlet or entrance time equal to 5 min., and the rain rates indicated by the limiting rain curve of 1899 for San Francisco, the amount of water on the ground at the end of successive intervals of 5 min. is shown in Column 5 of Table 5.

TABLE 5.—WATER ON THE GROUND DURING RAINFALLS OF EXTREME INTENSITY AT SAN FRANCISCO.

Based on Rain Intensities as determined in 1899, using the formula:

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

Successive periods: Duration, in minutes. (1)	Greatest possible total rain to the end of the period, in inches. (2)	Rain during each successive period, in inches. (3)	Intensity of rainfall, in inches per hour: or, approximately, second-feet per acre. (4)	Water on the ground at the end of each period, in cubic feet per acre. (5)	Greatest possible total rain to the end of each period, in cubic feet per acre. (6)	$G_T - G_{T+5}$ cubic feet per acre. (7)
0 to 5	0.150	0.150	1.795	360	540	....
5 to 10	0.219	0.069	0.828	166	790	195
10 to 15	0.275	0.056	0.672	134	1 000	32
15 to 20	0.321	0.046	0.552	112	1 170	22
20 to 25	0.364	0.043	0.516	103	1 320	9
25 to 30	0.404	0.040	0.480	95	1 470	8
30 to 35	0.441	0.037	0.444	89	1 610	6
35 to 40	0.476	0.035	0.420	84	1 730	5
40 to 45	0.509	0.033	0.396	79	1 830	5
45 to 50	0.541	0.032	0.374	75	1 970	4
50 to 55	0.571	0.030	0.360	72	2 080	3
55 to 60	0.600	0.029	0.348	70	2 180	2
60 to 120	0.906	0.306	0.306	61	3 290	....
120 to 180	1.158	0.252	0.252	50	4 200	....
180 to 240	1.396	0.238	0.238	48	5 090	....

TABLE 6.—Reduction Factors with Which to Multiply Maximum Rain Rates to Find Maximum Surface Run-Off Rates

The value of the constant, a, in $r = a\,I$

$$\text{When } I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}, \text{ or } I = \frac{60e}{\sqrt{t}}$$

Duration of the critical period, in minutes.	San Francisco. $b = 3.68$ $R = 0.60$ $e = 0.0775$					$b = 4$ $R = 0.65$ $e = 0.084$					$b = 8$ $R = 1.30$ $e = 0.169$					$b = 12$ $R = 1.95$ $e = 0.253$				
	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
5	1.00	0.91	0.81	0.71	0.61	1.00	0.91	0.82	0.73	0.64	1.00	0.96	0.91	0.87	0.82	1.00	0.97	0.94	0.91	0.88
10	1.00	0.88	0.76	0.64	0.51	1.00	0.89	0.77	0.66	0.54	1.00	0.94	0.88	0.83	0.77	1.00	0.97	0.93	0.89	0.85
15	1.00	0.86	0.71	0.57	0.42	1.00	0.87	0.74	0.60	0.47	1.00	0.93	0.87	0.81	0.74	1.00	0.96	0.92	0.87	0.83
20	1.00	0.84	0.68	0.53	0.37	1.00	0.86	0.72	0.56	0.43	1.00	0.93	0.86	0.79	0.72	1.00	0.96	0.91	0.86	0.81
30	1.00	0.83	0.66	0.49	0.31	1.00	0.85	0.69	0.53	0.37	1.00	0.92	0.84	0.76	0.69	1.00	0.95	0.90	0.85	0.79
40	1.00	0.82	0.64	0.46	0.27	1.00	0.84	0.67	0 51	0.34	1.00	0.92	0.83	0.75	0.67	1.00	0.95	0.89	0.84	0.78
50	1.00	0.81	0.63	0.44	0.25	1.00	0.83	0 66	0.49	0.31	1.00	0.91	0.82	0.74	0.65	1.00	0 94	0.88	0.83	0.77
60	1.00	0.81	0.62	0.43	0.23	1.00	0.82	0.65	0.47	0.29	1.00	0.91	0.82	0.73	0.64	1.00	0.94	0.88	0.82	0.76
90	1.00	0.80	0.59	0.38	0.18	1.00	0.82	0.63	0.44	0.25	1.00	0.91	0.82	0.72	0.63	1.00	0.94	0.88	0.81	0.75
120	1.00	0.79	0.58	0.37	0.16	1.00	0.81	0.62	0.42	0.23	1.00	0.91	0.81	0.71	0.62	1.00	0.94	0.87	0.80	0.74

Note: See notation on next page.

The absorption of water by the soil will vary directly with the proportional part of the surface that is pervious, and will also vary with the character of the soil and subsoil. It will be largest in districts having deep soils of the sand or gravel type that take an indefinite quantity of water freely, and will be least in districts with heavy clay soil or with a thin soil cover on impervious rock formations.

The rate at which the water thus deflected from a course across the surface of the ground will sink into the soil should be assumed to be a gradually decreasing rate. Soil will take water somewhat more readily at the beginning of a downpour than after a heavy rain has continued for some time.

This rate is determinable, and upon it will depend the relative amount of the rain which will from time to time, during the progress of a heavy rain, reach the storm-water inlets.

Data have not been accessible for an entirely satisfactory determination of reduction factors to be applied to rain rates to reduce the rain rates to surface run-off rates. The factors noted in Table 6, therefore, are to be considered as tentatively suggested. They are to be used until checked and corrected by further observations. They apply to soil conditions such that the rate of water absorption by the soil at the beginning of a critical period will be about $\frac{2}{3}$ in. per hour, decreasing gradually to about $\frac{1}{3}$ in. per hour at the end of 1 hour.

Notation.—The following notation is used in this paper:

A = Area, in acres, of the district above the point at which storm-water discharge is to be estimated;

T = End of a period of heavy rainfall;

T_m = Time at which the storm-water discharge is a maximum;

B = Beginning of a period of heavy rainfall;

B' = Time, t minutes preceding B;

t = Elapsed time, in minutes, from B to T;

t_m = Elapsed time, in minutes, from B to T_m;

R = Maximum rainfall in one hour; expressed in inches;

I = Maximum rain rate, in inches per hour (or, which is the same, second-feet per acre), for a period of time, t minutes in duration;

I_B = Maximum rain rate, in inches per hour, during t minutes preceding the time, B, during time, B' to B;

u_T = The rate of inflow into storm-water inlets during the time, B to T, expressed in cubic feet per second per acre;

u_B = The rate of inflow into storm-water inlets during the time, B' to B;

C = Volumetric capacity for storm-water (storage capacity) of all conduits, above the point at which discharge is to be estimated, expressed in cubic feet per acre;

C_B = Storm-water contents of the conduits at the time, B, expressed in cubic feet per acre;

C_T = Storm-water contents of the conduits at the time, T, expressed in cubic feet per acre;

$C_T - C_B$ = Therefore the augmentation of the water temporarily stored in the conduits during the time, B to T;

G_B = The water on the ground, on roofs, etc., and in the gutters, flowing toward the storm-water inlets, at the time, B, expressed in cubic feet per acre;

G_T = The water on the ground, etc., flowing toward the storm-water inlets at the time, T;

i = The time, in minutes, required by water to flow from the outermost portions of the area served by a storm-water inlet to the inlet;

G_{T+i} = The water on the ground, on roofs, etc., flowing toward the inlets, i minutes after the close of a critical period—a continuation of maximum rainfall conditions being assumed;

r = The surface run-off rate during the time, B to T, expressed in depth, in inches per hour, or in cubic feet per second per acre;

r' = The surface run-off rate during the time, B' to B, expressed in depth, in inches per hour, or in cubic feet per second per acre;

a = The constant in the equation $r = a\,I$ (and approximately $r' = a\,I_B$). This constant will have different values for different areas. For areas with impervious surface throughout, a is unity;

b = A constant appearing in the formula for rain intensity;

d = The mean discharge of storm-water, in cubic feet per second per acre, at the point for which storm-water discharge is to be estimated, during the time, B to T;

d_T = The storm-water discharge at the time, T, expressed in cubic feet per second per acre;

d_m = The maximum storm-water discharge at the time, T_m, expressed in cubic feet per second per acre;

d_B = The storm-water flow at the time, B, expressed in cubic feet per second per acre;

t_D = The time, in minutes, required for water to flow from the most remote portions of a district to the point at which discharge is to be estimated (including a time allowance, i, for flow to inlets);

S = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T;

S_m = The increase of the water quantity in temporary storage on the ground and in the conduits during the time, B to T_m;

q = The total depth of rain, in inches, during the time, B to T;

q' = The surface run-off, in inches, during the time, B to T;

e = A constant, appearing in the special case when the limiting rain curve is a parabola.

The Storm-Water Flow.—**The maximum quantity of rain, expressed in cubic feet, that may fall in the time, B to T, on one acre,** will be $60\ I\ t$. The portion of this water which will reach the storm-water inlets is $60\ r\ t$. This is the surface run-off per unit area, and must be equal to the total flow during the time, B to T, at the point for which discharge is to be estimated, plus the augmentation of the water storage on the ground and in the conduits during the same time.

$$60\ r\ t = 60\ a\ I\ t \quad \ldots\ldots (1)$$

$$60\ r\ t = 60\ d\ t + (C_T - C_B) + (G_T - G_B) \quad \ldots\ldots (2)$$

$$d = r - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots (3)$$

$$\text{or} \quad d = a\,I - \frac{(C_T - C_B) + (G_T - G_B)}{60\ t} \quad \ldots\ldots (4)$$

But if d is known, and if d_B can be determined, it will be possible to find the value of d_T on the assumption that

$$d = \frac{2}{3} d_T + \frac{1}{3} d_B \quad \ldots\ldots (5)$$

$$\text{From which} \quad d_T = \frac{1}{2}(3\,d - d_B) \quad \ldots\ldots (6)$$

The value assumed for d in Equation 5 results from the hypothesis that the discharge-curve for the time, B to T, approximates in shape a parabola, tangent to a horizontal line at T and passing through the point B, as shown in Fig. 7.

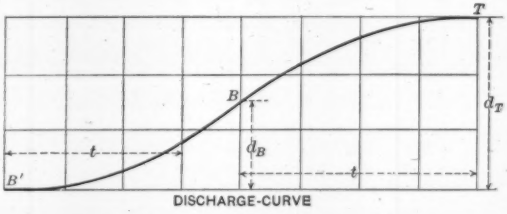


FIG. 7.

But the value of d_B can be approximated on the further assumption that only the rainfall in the t minutes preceding the time, B, need be taken into account. For the greatest possible rain intensity preceding B, the value of d_B, without too great error, may be called equal to the mean run-off rate during the t minutes preceding B. That is to say:

Because, approximately, $r' = a\, I_B$ ..... (7)

Therefore, near enough, $d_B = r' = a\, I_B$ ..... (8)

And from Equation 6, $d_T = \frac{1}{2}(3\, d - a\, I_B)$ ..... (9)

It appears from the above that the value of d increases with increasing values of C_B. It increases in value, also, for decreasing values of C_T. (See Equation 4.) Therefore the rain which will give d maximum value must be so distributed to the time preceding and immediately following the time, B, that the conduits will be as full of water as possible at the beginning, and at as low a stage as possible at the end, of the critical period. With this fact the type of rain storm becomes known which will make the mean discharge throughout a critical period a maximum.

The rain must evidently fall preceding the time, B, with constantly increasing intensity, and during the time, B to T, with constantly decreasing intensity.

It follows that the greatest possible rain intensity, at a time instant immediately preceding B, will be the same as the greatest possible rain intensity at the time, T. Consequently, for the most unfavorable rain conditions, the value of G_B will be equal to G_{T+i} (the inflow time being i minutes).

Equation 3 may now be written

$$d = r - \frac{(C_T - C_B) + (G_T - G_{T+i})}{60\,t} \quad \ldots\ldots\ldots (10)$$

but $$S = C_T - C_B + G_T - G_{T+i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

therefore $$d = r - \frac{S}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The values of G_T and G_{T+i} can be calculated for any rain curve, as has already been shown. The value of d in Equations 3 and 10 and of d_T in Equation 6 are dependent, not alone upon the intensity of the rain, but also upon the duration of the extreme rain conditions, and upon the volumetric increase of the water in the conduits and on the ground during the critical period.

It may be stated in passing that, for the inlet ends of the conduits, $C_T - C_B = 0$. Therefore, from Equation 10, for the special case in which $d = u$ and $d_T = u_T$

$$u = r - \frac{G_T - G_{T+i}}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

calling $i = 5$

$$u = r - \frac{G_T - G_{T+5}}{60\,t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

and from Equation 6

$$u_T = \frac{1}{2}\,(3\,u - r') \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

It may be inferred from Equation 9,

$$d_T = \frac{1}{2}\,(3\,d - a\,I_B),$$

that the value of d_T will follow closely the value of d; that, therefore, the same type of rain storm which will give the mean discharge during a critical period maximum value will also make the discharge at the time, T, maximum. It is reasonably certain that this is the case, as will be shown later in the presentation of a graphical solution of the problem. This fact is not demonstrated by Equation 9, because C_B is at maximum value when I_B is at maximum value, while d_T increases with the increasing value of C_B and with the decreasing value of I_B. The graphical illustration will show that in all ordinary cases the effect of changing values of C_B upon the value of d_T outweighs the effect of the corresponding changes in the rain rate, I_B. But the introduction of the variable, I_B, into the calculation of d_T from d makes it apparent that the time at which d is maximum is not

necessarily the time at which d_T is maximum. These times, in fact, are not coincident.

The least satisfactory part of the foregoing demonstration lies in the approximation of a value for d_B and in the assumption that the maximum discharge can be computed by Equation 9 from the mean discharge during the critical period and the discharge at the beginning of the period.

Nor is it, as a matter of course, strictly correct, except in the case of impervious areas, to assume that the relation of r' to I_B is the same as the relation of r to I. This assumption is probably near enough to the truth for all practical purposes.

The true value of d_T, therefore, may depart to some extent from the value determined, as explained, from d and d_B. The room for such departure will decrease as the values of d_B and d approach each other; it will be greatest when $d_B = 0$; that is, the opportunity for a departure from the correct value is greatest in the case of no rain preceding the critical period. It is quite possible that for this special case some other expression than Equation 9 can be found that will better express the relation between d, d_B, and d_T; but the further investigation of this special case would only be of interest if it were desired to deduce formulas for application to types of rain storms other than those which produce the maximum discharge.

To estimate the values of C_T and C_B let it be assumed that a first approximation of conduit sizes by any method has established a first value of C. The value of C when the conduits are to carry water other than that due to rain will not, as a matter of course, be the entire volumetric contents of the conduits, but that portion thereof which is available for the temporary storage of rain water. Call t_m equal to t_D. This fixes tentatively the position of T in the time scale. The limiting rain curve furnishes the value of I and of I_B, and, by application of the proper reduction factor, the values of r and of r' are found from $r = a\,I$ and $r' = a\,I_B$.

It is now possible to estimate the maximum value of C_B. This will result, as already explained, if the rain storm is of such a character that, preceding the time, B, there is a rainfall of constantly increasing intensity. See Fig. 8. The flow at the time, B, can now be calculated; but the ratio of this flow to the flow at the time, T, may be, preliminarily, considered the same as the ratio of I_B to I.

With the aid of this ratio, on the assumption that the space occupied by water in the conduits is proportional to the flow (the velocities being practically the same for half full and for full or nearly full stages) a first approximation of C_B is possible. The first approximate value of C_T is C. That is, all conduits are assumed to be full at the time, T.

The value of d resulting from this assumption will be too small, because, even under the most unfavorable conditions, C_T will always be somewhat less than C.

The value of d_T follows by the use of Equation 9.

The conduit sizes originally entering into the calculation can now be revised, and the values of C_B and C_T are again determined. It is now desirable to make the estimate of C_T with greater precision than for first approximation purposes. This can readily be done, because, at the time, T, the conduits are full at the point for which storm-water flow is to be estimated and the rate of flow at all upper ends of conduits can be estimated from the rain rate which prevailed in the last i minutes preceding the time, T.

It will ordinarily be found convenient to call

$$C_T = KC \quad \ldots\ldots(16)$$

where K is a coefficient to be ascertained for each area under consideration. The value of K will approach unity for small districts.

From Equations 9 and 10,

$$d = r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}$$

$$\text{and} \quad d_T = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t} - \frac{G_T - G_{T+5}}{60\,t}\right) - r'\right].$$

For all cases in which t exceeds 15 min., $G_T - G_{T+5}$ will have so small a value that the term containing this expression may be neglected, therefore, for $t > 15$,

$$(d_T)_{t > 15} = \frac{1}{2}\left[3\left(r - \frac{K\,C - C_B}{60\,t}\right) - r'\right] \quad \ldots\ldots(19)$$

$$\text{or} \quad (d_T)_{t > 15} = \frac{1}{2}\left[3\left(a\,I - \frac{K\,C - C_B}{60\,t}\right) - a\,I_B\right] \quad \ldots\ldots(20)$$

Numerical Illustrations.—Let it be required to determine the capacity of the outfall sewer for a business district in San Francisco, with impervious surface throughout, the area of the district being 1 000 acres. Suppose that the district has such shape and such surface slope that it will take water 50 min., including a 5-min. entrance

allowance, to flow from the most remote portion thereof to the point at which the conduit capacity is to be determined. Temporary storage of water on the ground's surface, because $t_D > 15$, may be neglected ($G_T - G_{T+5} = 0$).

The maximum rain rate in San Francisco for 50 min., when $I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$ (see Table 2), is 0.65 in. per hour, or 0.65 sec-ft. per acre

For an impervious surface, $a = 1.00$; therefore $r = I$, and $r' = I_B$.

A first approximation of the storm-water discharge can be made by the method heretofore described as in use in San Francisco.

By this method,

$$d_m = 0.75 \times 0.65 = 0.488 \text{ sec-ft. per acre.}$$

Let it now be assumed that calculations for a number of points in the district have established conduit sizes having a volumetric capacity for storm-water of 1 500 cu. ft. per acre.

Let it further be supposed that calculations already made have established $K = 0.8$.

An examination of the limiting rain curve of the type recommended for use in San Francisco will show that during a period of time, t minutes in duration, immediately preceding or following a critical period of the same duration, the greatest possible rainfall will be, in round numbers, one-half of the rain during the critical period. For approximation purposes, therefore, the quantity of water in the conduits at the beginning of a critical period may be taken at one-half of the amount in the conduits at the close of the period.

Approximately $\quad C_B = \frac{C_T}{2},$

but $\quad C_T = K\,C = 0.8\,C,$

therefore, $\quad C_T - C_B = 0.4\,C = 600.$

The limiting rain curve for San Francisco, based on $I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$, Fig. 6, indicates that in the 50 min. preceding the time, T, there may be a rainfall aggregating 0.539 in.; that is, $I = 0.647$ in. per hour. The rainfall in 50 min. preceding the critical period cannot, as shown by the rain curve, exceed 0.272 in., because the total rain in 100 min. cannot exceed 0.811 in.

The rainfall, 0.272 in. in 50 min., represents a rain intensity of 0.326 sec-ft. per acre.

Consequently, $r = a\,I = \quad 0.647$ (a being unity)

$r' = a\,I_B = 0.326$

From Equation 19, $d_T = \frac{1}{2}\left[3\left(0.647 - \frac{600}{3\,000}\right) - 0.326\right]$

$= 0.508$ sec-ft. per acre.

$A\,d_T = 508$ sec-ft.

As this value is somewhat larger than the first approximation, it may now be necessary to correct the value of C. For the purpose of this illustration, it will be supposed that no correction is necessary.

Other values of t are now to be tried, in order to find that value of t which will give d_T maximum value.

For $t = 40$ min.

$r = 0.711$

$r' = 0.353$

$d_T = \frac{1}{2}\left[3\left(0.711 - \frac{600}{2\,400}\right) - 0.353\right]$

$= 0.515$

$A\,d_T = 515$ sec-ft.

For $t = 30$ min.

$r = 0.806$

$r' = 0.392$

$d_T = \frac{1}{2}\left[3\left(0.806 - \frac{600}{1\,800}\right) - 0.392\right]$

$= 0.513$

$A\,d_T = 513$ sec-ft.

The critical period is evidently about 40 min., and the maximum storm-water discharge, or required conduit capacity, will be about 515 sec-ft.

As a second illustration, let it be required to ascertain the storm-water discharge from a district, 1 000 acres in area, in which only 250 acres of the surface are impervious. Here, again, the time, t_D, is to be approximated from probable conduit gradients. Suppose $t_D = 50$ min.

The value of a, to be used in the equations $r = a\,I$ and $r' = a\,I_B$, will be found in Table 6, that is $a = 0.44$.

Suppose that, by preliminary estimate, $C = 800$, and that $K = 0.8$.

Again accepting the approximate value, $C_B = \frac{C_T}{2}$, makes $C_B = 320$.

For $t = t_D = 50$ min.

$I = 0.647$

and $I_B = 0.326$

$r = 0.44\, I = 0.285$

$r' = 0.44\, I_B = 0.143$

$$d_T = \frac{1}{2}\left[3\left(0.285 - \frac{320}{3\,000}\right) - 0.143\right]$$

$$= 0.196$$

$A\, d_T = 196$ sec-ft.

For $t = 40$ min.

$I = 0.711$

$I_B = 0.353$

$r = 0.44\, I = 0.313$

$r' = 0.44\, I_B = 0.155$

$$d_T = \frac{1}{2}\left[3\left(0.313 - \frac{320}{2\,400}\right) - 0.155\right]$$

$$= 0.192$$

$A\, d_T = 192$ sec-ft.

For $t = 60$ min.

$I = 0.599$

$I_B = 0.306$

$r = 0.44\, I = 0.264$

$r' = 0.44\, I_B = 0.135$

$$d_T = \frac{1}{2}\left[3\left(0.264 - \frac{320}{3\,600}\right) - 0.135\right]$$

$$= 0.195$$

$A\, d_T = 195$ sec-ft.

The critical time is evidently about 50 min., and the required capacity about 196 sec-ft.

It must be remembered that the values of the reduction factor, a, noted in Table 6, are based on a specific rate of water absorption by the soil. They are to be suitably modified whenever, for any district, other rates of water absorption by the soil shall have been determined.

The increase of water contents of the conduits, when conduits are already constructed, should be calculated from the estimated water stages whenever this is practicable, instead of using the above indicated approximation method.

How the value of t_m may be found, when the storage increase during a critical period is known, will be explained later.

Graphical Solution.—The method of estimating the required capacity of storm-water conduits just illustrated becomes of importance, first, because it is based on sound principles and can be made applicable to districts of any shape, character, and size; and second, because it enables the problems to be solved by simple graphical methods, as will now be explained.

With the mass curve of rainfall for extreme conditions of precipitation as a guide, there is to be constructed for any area under consideration a second integral or mass curve representing the maximum amount of surface run-off during the progress of a downpour of the extreme type. The new curve will be the rain curve modified by the appropriate reduction factor, a, to be found in Table 6. The new curve, for convenience, may be called the surface run-off curve.

The surface run-off curve for areas with impervious surface throughout will coincide with the limiting rain curve, because, for the impervious surface, the factor, a, is unity and $r = I$. The San Francisco surface run-off curve for impervious surface, based on rain intensities expressed by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

(which will closely approximate the rain curve of 1899), is shown in Fig. 9.

The time scale in this diagram indicates minutes subsequent to the time, B. The ordinates represent, in cubic feet per acre, the water which will ultimately reach the storm-water inlets.

The total run-off during any time period is the difference between the ordinates at the beginning and at the end of the period. Some of this water, however, is still flowing across the surface toward the inlets at the end of the period.

The surface run-off curve for an area having a surface which is not impervious throughout can be obtained from the curve for impervious areas by multiplying the ordinates by the proper reduction factors.

Suppose now that the limiting surface run-off curve be platted in a system of co-ordinates with the point, B, at the beginning of the co-ordinates (Fig. 8).

Let T be located on this run-off mass curve so that B to T on the

time scale is the actual critical period for the point of the conduit under consideration.

Make $T\,D' = S$

the value of S being $S = C_T - C_B + G_T - G_{T+t}$ .....(11)

then $d = \dfrac{D'\,F}{60\,t}$

The storm-water discharge at the time, B, as already explained, which will make d a maximum, will be the discharge due to a maximum possible fall of rain in the time, t minutes, preceding B. The rainfall in this time must be one of increasing intensity. The run-off curve, therefore, will take the position as shown, from B'' to B.

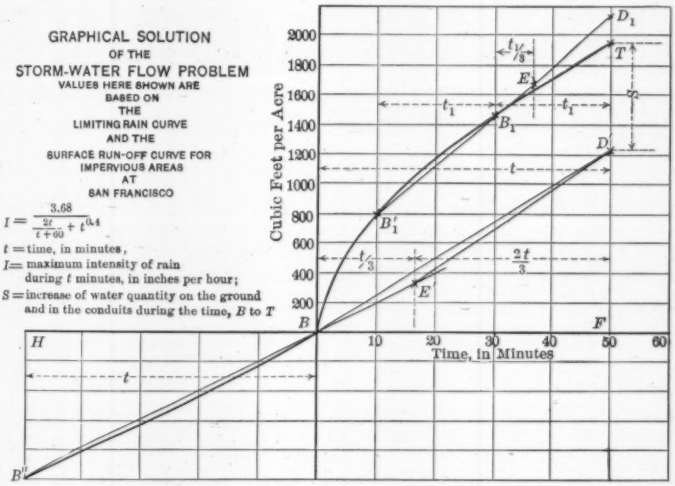


FIG. 8.

The discharge of storm-water at the time, B, may be taken, without material error, as equal to the mean surface run-off rate during the time, B'' to B, provided, of course, that t be not at material variation from t_D. Therefore,

$$d_B = \frac{B''\,H}{60\,t}.$$

The mass curve of storm-water discharge after the time, B, will therefore be represented by a curve to which the line, $B''\,B$, is tangent at B, and which will gradually rise to the point, D'. The inclination

of the tangent to this curve at the point, D', will represent the maximum flow at the time, T.

The shape of this curve will probably be such that its change of direction is somewhat more rapid at the time, B, at which the downpour is most intense, than at the time, T, at which the flat top of the flood wave in the conduit has reached the point for which discharge is being estimated. It will be reasonable to assume, if B'' B be

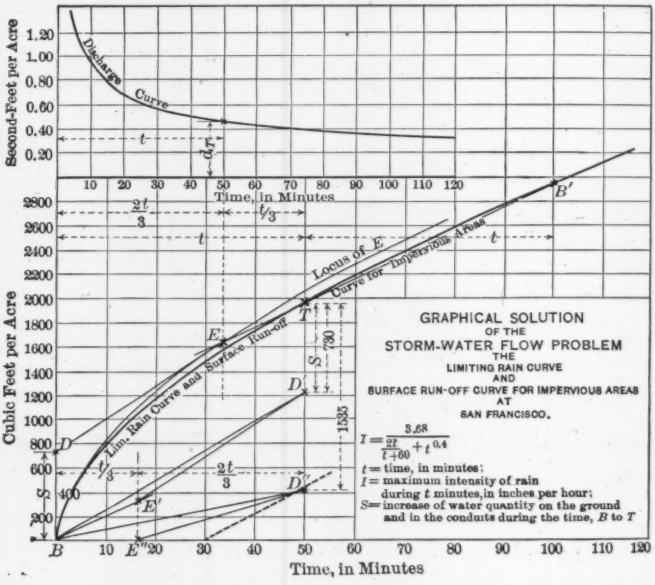


FIG. 9.

prolonged to the point, E', on the time ordinate, one-third of the way from B to T, that the line, E' D', will be a close approximation to the inclination of the curve at the point, D'. This is in conformity with the assumption heretofore made that

$$d = \frac{2\,d_T}{3} + \frac{d_B}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

By assuming a new position for the point, T, with corresponding modification of the curve, B'' to B, a new position of the point, E', will be found. It thus becomes possible, by determining successive locations of the points, E' and D', to find the particular value of t

which will give the line, E' D', its greatest inclination; which will, in other words, for the known value of S, give d_T maximum value.

The same result may be obtained in a much simpler way. Suppose the value of S to be scaled off upward from B in the diagram, Fig. 9, establishing the point, D. Let B' be the point on the run-off mass curve for a time period equal to 2 t. Then the line, B' T, of this diagram will be parallel to the line, B'' B, of the diagram, Fig. 8. If this line, B' T, prolonged cuts the time ordinate lying two-thirds of the way from B to T at E, then D E will be parallel to the line, E' D', of the diagram, Fig. 8, and the inclination of the line, D E, will give the desired value of d_T.

For every possible location of T a new position of E will be found. The successive positions of E will lie upon a curve which, as will readily be seen, is always the same no matter what the value of S may be. Therefore, to find the maximum value of d_T, it will only be necessary to draw a tangent from the point, D, Fig. 9, to the E-curve.

The point of tangency thus found will be distant from B on the time scale two-thirds of the time, t_m; and the inclination of the tangent will be the greatest possible value of d_T.

For the limiting rain curve of San Francisco, based on

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

and impervious surfaces ($a = 1$), the resulting locus of the point, E, or the E-curve, is shown in Fig. 9.

Suppose, now, that a value of $C_T - C_B$ has been approximated. Add to this $G_T - G_{T+5}$, obtained from Table 5, on the supposition that $i = 5$ and that t_m will be about equal to t_D.

Scale upward from B the value of S thus obtained, and draw D E tangent to the E-curve at E.

Then d_T = inclination of line, D E,

and t_m = one and one-half times the scaled time from B to E.

It will be convenient to complete the diagram by platting to suitable scale the differentials of the E-curve. A third curve will thus be obtained which may be called the discharge curve. This curve will give at once the value of d_m for any value of t_m.

By calling $t_m = t_D$, a quick first approximation of the value d_m can thus be made.

It follows from the preceding demonstrations that for every mass

curve of surface run-off, on the hypotheses as made, there is a definite inter-relation between the values of S_m, t_m, and d_m. This inter-relation can be determined from the diagram, and can be tabulated.

Ordinarily, the fact most readily ascertainable for any district is the value of t_D, which (in the case of urban storm-water flow problems) may be accepted as a first approximation of the value of t_m. In tabulating the information obtained by diagram it will, therefore, be convenient to note regular time intervals in the first column.

The values noted in Columns 2, 3, and 4 of Table 7 are based on the limiting rain curve for San Francisco as determined by the formula,

$$I = \frac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$$

and are applicable to areas with impervious surface throughout.

The corresponding columns in Table 8 contain values based on the same limiting rain curve with reductions due to water absorption by soil when only 25% of the surface is impervious. (For the reduction factors used see Table 6.)

TABLE 7.—Storm-Water Flow in Its Relation to the Duration of the Critical Time Period, and to the Augmentation of the Water Quantity Temporarily Stored in the Conduits.

For areas with impervious surface throughout. For San Francisco.

Based on $I = \dfrac{3.68}{\frac{2\,t}{t+60} + t^{0.4}}$.

Duration of critical period, t_m, in minutes.	Augmentation of storage in the conduits, $C_T - C_B$, in cubic feet per acre.	By Diagram.		Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
		Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	Discharge, d_m, in second-feet per acre.	
(1)	(2)	(3)	(4)	(5)
5	5	200	1.25	1.34
10	270	300	0.93	0.99
15	365	385	0.78	0.82
20	440	450	0.69	0.71
30	545	555	0.57	0.60
40	635	640	0.51	0.54
50	725	730	0.46	0.49
60	800	800	0.43	0.45
70	870	870	0.40	0.43
80	930	930	0.38	0.41
90	985	985	0.36	0.38
120	1 120	1 120	0.32	0.34

TABLE 8.—STORM-WATER FLOW, IN ITS RELATION TO THE DURATION OF THE CRITICAL PERIOD AND TO THE AUGMENTATION DURING SUCH PERIOD OF THE WATER QUANTITY TEMPORARILY STORED IN THE CONDUITS.

For areas with 25% of the surface impervious. For San Francisco.

$$\text{Based on } I = \frac{3.68}{\frac{2t}{t+60} + t^{0.4}}.$$

Duration of critical period, t_m, in minutes.	Augmentation of the storage in the conduits, $C_T - C_B$, in cubic feet per acre	By Diagram.		Discharge, by method used in 1899, calling $t_m = t_D$, in second-feet per acre.
		Augmentation of storage on the ground and in the conduits, S, in cubic feet per acre.	Discharge, d_m, in second-feet per acre.	
(1)	(2)	(3)	(4)	(5)
5	5	140	0.89	0.63
10	170	190	0.60	0.46
15	205	220	0.45	0.39
20	230	240	0.37	0.34
30	265	270	0.28	0.28
40	290	295	0.24	0.25
50	320	320	0.20	0.23
60	345	345	0.19	0.21
90	375	375	0.14	0.18
120	415	415	0.12	0.16

Referring again to Fig. 8, it remains to be shown that no rain condition preceding the time, B, can be assumed which will give the line, $E'D'$, greater inclination than resulting from that which has already been declared to be the most unfavorable.

Assume the extreme case of no rain preceding the time, B. In this case there will be no storm-water in the conduits and no water on the ground at the time, B. Consequently, $C_B = 0$, and $G_B = 0$.

From Equation 11, $S = C_T + G_T$;

but, as has already been shown, the maximum value of C_B for extreme rain intensity preceding the time, B, is about one-half of C_T; therefore, approximately, for this condition,

$$C_T = 2\,(C_T - C_B),$$

where C_B represents volumetric water contents of the conduits due to maximum rainfall conditions prior to the time, B.

For the special case shown in the diagram,

for $t = 50$,	$C_T - C_B =$	730,
therefore	$C_T =$	1 460.
From Table 5,	$G_T =$	75
consequently	$S =$	1 535.

If the critical time were the same for the condition of no rain preceding the time, B, as for the condition of heavy rain preceding B, then it would be proper to scale this value of S downward from T in Fig. 9, and it would at once be apparent, because the inclination of E'' D'' is less than the inclination of E' D', that the most unfavorable assumption is that of heavy rain preceding the critical period. But the durations of the critical periods are not the same in the two cases. On the same hypothesis on which Equation 5 is based, the critical period for the case of no preceding rain is of nearly twice the duration as for the case of heavy preceding rain. For the special values above noted, one-third of the critical period would be about 30 min., and the corresponding value of d'_m would be the inclination of the broken line (Fig. 9) rising from the 30-min. point of the base line.

This, also, is less than that resulting from the originally assumed unfavorable conditions. The same result can be shown for any other value of t and for any other rain intensities preceding the time, B, less than those originally assumed as the least favorable.

The conclusion relating to the most unfavorable distribution of rain heretofore announced appears therefore to be correct. The rain rates that should be assumed as prevailing before the beginning of the critical period are the greatest rates that are possible, the rain being such that it falls with gradually increasing intensity until the time, B.

The comparison of the results by the new method of computation with those obtained by the method of 1899, as shown in Table 7, indicate reasonably close agreement for areas with impervious surface.

The comparison in Table 8 for areas which are only in small part impervious indicates a greater range of unit values for the new method than result from the older method of calculation. The method of 1899 probably gives too small values for small areas which are only partly impervious.

It may be repeated that water quantities in the conduits at the times, B and T, can ordinarily be calculated with a sufficient degree of approximation. At the time, T, the main conduit is flowing at capacity at the point for which storm-water flow is to be estimated, and the inlets on all the conduit branches are at the same time receiving water at the run-off rate which corresponds to the last few minutes of

the critical time period. It is possible, therefore, to determine to what proportion of their capacity the conduits are charged at their inlet ends. With this information for the upper ends of conduits, and the knowledge that the conduits are full near the point under consideration, a close approximation of the water stage at the time, T, is possible, and C_T may be calculated.

By a similar line of reasoning, the stage of water in the conduits can be determined for the time, B, and the value of C_B can also be calculated.

But for first approximation purposes it will be well to establish the values of K for increasing areas. Then call the ratio of the water stored in the conduits at the time, B, to the water stored at the time, T, equal to the ratio of the mean run-off during the t_D minutes preceding B to the mean run-off in the t_D minutes after B. For San Francisco this ratio, as already stated, may be called as one to two.

$$\text{That is,} \qquad C_B = \frac{C_T}{2};$$

$$\text{but} \qquad C_T = K\,C,$$

$$\text{therefore} \qquad C_B = \frac{K\,C}{2},$$

$$\text{and } C_T - C_B = \frac{K\,C}{2}.$$

To the value of $C_T - C_B$ thus or otherwise determined add the value of $G_T - G_{T+i}$, as found in Column 7 of Table 5 for $i = 5$. The sum will be the value of S to be used, as already explained.

When this method of determining storm-water flow is applied to a conduit system already constructed, it may happen that, owing to excessive or deficient storage capacity in the conduits, there will be wide divergence in the values of t_m and t_D.

The value of t_m may be considerably less than t_D if the conduits are too small, if they are surcharged at critical times; and it may be materially larger than t_D if along the line of the conduits there is unusual reservoir space to be filled before the maximum stage is attained. It will be nearly twice as great as t_D for the special case of no rain preceding the critical period.

The graphical method of determining the storm-water flow is applicable to any limiting mass curve of surface run-off. It is applicable therefore to any such curve obtained by applying a reduc-

tion factor to the run-off rates from areas with impervious surfaces.

But when the problem is presented of finding the storm-water flow for districts having surfaces which are not impervious throughout, it will not be necessary to construct new curves. The original curve for impervious conditions throughout may be used to solve any such problem.

When the value of $S = C_T - C_B + G_T - G_{T+t}$ has been determined for an area of any surface character, divide this value by the factor, a (to be found in Table 6), and proceed with the modified value of S thus obtained ($S' = \frac{S}{a}$), as though the area had an impervious surface throughout. There will thus be obtained a modified value of d_T, which may be called d'_T, such that

$$d'_T = \frac{d_T}{a},$$

$$\text{therefore,} \qquad d_T = a\, d'_T.$$

That is to say, the value thus obtained from the curves or from the tables is to be multiplied by the same reduction factor before used, to give the value of d_T.

New Formula and Discharge Tables.—A study of the limiting rain curve of 1899 for San Francisco has led to the adoption of a formula, as already stated, which, it is believed, will be found generally applicable for the determination of rain intensities for all periods of time that come under consideration in urban run-off problems.

In its general form the formula may be written

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \qquad \text{.....(21)}$$

Here b is a constant for any locality, and t is the duration of the rain, in minutes.

This formula gives the value of I, in inches per hour, or, which is the same thing, in cubic feet per second per acre.

For San Francisco, $b = 3.68$ will give values closely agreeing with those determined in 1899. (See Table 3.)

For Atlantic Slope conditions, ordinarily, $b = 8$ to 12.

Wherever the maximum rain intensity for 1 hour is known, the value of b can be determined. It will generally be well, however, provided data are adequate, to base the value of b on determinations from the maximum rain intensities for 10 min., 30 min., and 1 hour. In most cases the use of an average value from these three determinations should satisfy every requirement.

The rain intensities resulting from $b = 3.68$, 4, 8, and 12, in this formula, have already been noted in Table 3.

Whether the expression recommended, or any other expression in the general form

$$I = c\, f\,(t) \qquad (22)$$

be used in estimating the maximum rain intensity during the time, t, the fact is noteworthy that for all possible values of c or of b in the other case, the corresponding ordinates of the resulting mass curves will be proportional to the values of c or b that have been used.

This fact makes it possible to establish by a single diagram (for impervious surface conditions), base values of storage augmentation and of storm-water flow, being for the special case of $b = 1$ or of $c = 1$.

From such base values the storage and discharge for other values of b or of c, and also for any condition of the surface, can then be found by multiplication with the factors, ab or ac, as the case may be.

Table 9 contains the base values of S_m, d_m, and $(C_T - C_B)$, that is, $(S_m)_{\substack{a=1\\b=1}}$, $(d_m)_{\substack{a=1\\b=1}}$, and $(C_T - C_B)_{\substack{a=1\\b=1}}$, for the type of rain curve herein recommended, as determined by diagram.

The values in the last column of Table 9 conform closely to a curve the equation for which is

$$y = \frac{0.71}{\frac{2\,t_m}{t_m + 60} + t_m^{\,0.4}} \qquad (23)$$

This is the value of d_m for the special case in which $a = 1$ and $b = 1$. For any value of a and of b, therefore,

$$d_m = \frac{0.71\,a\,b}{\frac{2\,t_m}{t_m + 60} + t_m^{\,0.4}} = 0.71\,a\,I \qquad (24)$$

As it frequently will be desirable to compute the discharge directly from the maximum rainfall in 1 hour, the value of b may be found in terms of R and substituted.

For $t = 60$, there will be $I = R$.

$$I = \frac{b}{\frac{2\,t}{t + 60} + t^{\,0.4}} \qquad (21)$$

$$R = \frac{b}{6.14} \qquad (25)$$

$$b = 6.14\,R \qquad (26)$$

Therefore, from Equation 24,

$$d_m = \frac{4.36\, a\, R}{\frac{2\, t_m}{t_m + 60} + t_m{}^{0.4}} = 4.36\, a\, R \left(\frac{I}{b}\right) \quad \ldots\ldots\ldots\ldots (27)$$

TABLE 9.—BASE VALUES OF d_m AND $C_T - C_B$; WHEN $a = 1$ AND $b = 1$; IN THE FORMULA FOR SURFACE RUN-OFF:

$$r = a\, I = \frac{a\, b}{\frac{2\, t}{t + 60} + t^{0.4}}$$

All values determined by diagram.
(Subject to correction.)

Duration of the critical period, t_m, in minutes.	Total storage increase, $(S_m)\ a = 1,\ b = 1$, in cubic feet per acre.	Increase of storage in the conduits, $(C_T - C_B)\ a = 1,\ b = 1$, in cubic feet per acre.	Discharge, $(d_m)\ a = 1,\ b = 1$, in second-feet per acre.
5	54	0	0.340
10	82	73	0.252
15	105	100	0.212
20	122	120	0.187
30	151	150	0.156
40	174	170	0.139
50	198	197	0.125
60	218	218	0.116
90	268	268	0.097
120	305	305	0.087

In the practical application of this formula, a should be considered constant for each area, as already explained. Its value is directly dependent upon the rate of water absorption by exposed soil surfaces during heavy rains. The value of a for any value of b and any value of t, therefore, can be determined by experiment. For surfaces impervious throughout, the value of a is unity.

Some other values of a preliminarily suggested for use in this formula are to be found in Table 6.

The foregoing discharge formula will be directly applicable to the determination of the maximum discharge when, for any district under consideration, the value of t which will make d_T a maximum is known.

As a first approximation, without great error, t_m may be called equal to t_D. Consequently, a first approximation of d_T by the formula is possible whenever the time can be estimated that it will

take water to flow from the most remote portions of a district to the point at which discharge is being estimated.

It can readily be seen, when

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}} \quad \ldots\ldots(21)$$

$$\text{that } (G_T - G_{T+t}) = a\,b\,[G_T - G_{T+t}]_{\substack{a=1\\b=1}} \quad \ldots\ldots(28)$$

$$\text{and that } \quad (C_T - C_B) = a\,b\,[C_T - C_B]_{\substack{a=1\\b=1}} \quad \ldots\ldots(29)$$

$$\text{consequently, also, } \quad S = a\,b\,(S)_{\substack{a=1\\b=1}} \quad \ldots\ldots(30)$$

From the values given for $(S_m)_{\substack{a=1\\b=1}}$ in Table 9, for the special case of $i = 5$, the relation between t_m and $(S_m)_{\substack{a=1\\b=1}}$ can be reduced to formula. It will be found that, with a sufficient degree of accuracy for all values of t_m which are likely to arise in connection with urban problems,

$$(S_m)_{\substack{a=1\\b=1}} = 27.5\,\sqrt{t_m} \quad \ldots\ldots(31)$$

$$\text{therefore } \quad S_m = 27.5\,a\,b\,\sqrt{t_m} \quad \ldots\ldots(32)$$

$$\text{and } \quad t_m = \left(\frac{S_m}{27.5\,a\,b}\right)^2 \quad \ldots\ldots(33)$$

From Equations 11 and 32,

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - (G_T - G_{T+t}) \quad \ldots\ldots(34)$$

$$C_T - C_B = 27.5\,a\,b\,\sqrt{t_m} - a\,b\,(G_T - G_{T+t})_{\substack{a=1\\b=1}} \quad \ldots\ldots(35)$$

and when $i = 5$,

$$C_T - C_B = a\,b\left[27.5\,\sqrt{t_m} - (G_T - G_{T+5})_{\substack{a=1\\b=1}}\right] \quad \ldots\ldots(36)$$

It has already been shown that

$$b = 6.14\,R \quad \ldots\ldots(26)$$

Consequently, from Equation 32,

$$S_m = 169\,a\,R\,\sqrt{t_m} \quad \ldots\ldots(37)$$

and from Equation 36,

$$C_T - C_B = a\,R\left[169\,\sqrt{t_m} - 6.14\,(G_T - G_{T+5})_{\substack{a=1\\b=1}}\right] \quad \ldots\ldots(38)$$

Likewise, from Equation 37,

$$t_m = \left(\frac{S_m}{169\,a\,R}\right)^2 \quad \ldots\ldots(39)$$

and, convenient for purposes of preliminary approximation, from Equations 27 and 37,

$$d_m = \frac{S_m}{38.8\sqrt{t_m}}\left(\frac{1}{\frac{2\,t_m}{t_m+60}+{t_m}^{0.4}}\right) \quad \ldots\ldots\ldots\ldots (40)$$

TABLE 10.—STORM-WATER FLOW.

FOR $b = 4$, IN THE FORMULA, $I = \dfrac{b}{\frac{2\,t}{t+60}+t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\,a\,b}{\frac{2\,t_m}{t_m+60}+{t_m}^{0.4}} = \frac{4.36\,a\,R}{\frac{2\,t_m}{t_m+60}+t^{0.4}}$$

$$S_m = 27.5\sqrt{t_m}$$

and $C_T - C_B = S_m - (G_T - G_{T+5})$

$b = 4$ and $R = 0.65$.

Duration of critical period, t_m, in minutes.	ALL IMPERVIOUS.		75% IMPERVIOUS.		50% IMPERVIOUS.		25% IMPERVIOUS.		NONE IMPERVIOUS.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	35	1.38	35	1.26	30	1.13	30	1.01	25	0.88
10	310	1.02	280	0.91	250	0.79	210	0.67	165	0.55
15	400	0.85	350	0.74	295	0.63	240	0.51	190	0.40
20	480	0.74	410	0.63	345	0.53	255	0.42	205	0.32
30	600	0.62	500	0.53	410	0.43	315	0.33	220	0.23
40	690	0.55	580	0.46	460	0.37	350	0.28	235	0.185
50	780	0.50	650	0.42	510	0.33	380	0.25	240	0.155
60	850	0.46	700	0.38	550	0.30	400	0.215	245	0.135
90	1 050	0.39	860	0.32	660	0.245	460	0.170	260	0.098
120	1 210	0.35	980	0.28	740	0.215	510	0.145	275	0.080

When, therefore, the increase of water storage during a critical time period can be determined, the duration of this period can be found from Equations 33 or 39, and d_m can be estimated by formula, Equations 24 or 29; or the value of d_m can be taken from Tables 10, 11, and 12, or from the general table, Table 13. Or, the value of d_m may be estimated directly from the value of S_m and an approximate value of t_m by the use of Equation 40. In this last case the estimated discharge is to be treated as an approximation until the value of t_m has been verified.

TABLE 11.—STORM-WATER FLOW.

FOR $b = 8$, IN THE FORMULA, $I = \dfrac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\,a\,b}{\dfrac{2\,t_m}{t_m+60} + t_m^{\,0.4}} = \frac{4.36\,a\,R}{\dfrac{2\,t_m}{t_m+60} + t^{0.4}}$$

$$S_m = 27.5\,a\,b\,\sqrt{t_m}$$

and $C_T - C_B = S_m - (G - G_{T+5})$

$$b = 8 \text{ and } R = 1.30.$$

Duration of critical period, t_m, in minutes.	ALL IMPERVIOUS.		75% IMPERVIOUS.		50% IMPERVIOUS.		25% IMPERVIOUS.		NONE IMPERVIOUS.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	70	2.77	70	2.65	65	2.52	65	2.40	60	2.27
10	620	2.03	590	1.91	550	1.79	520	1.68	480	1.56
15	810	1.70	760	1.58	710	1.48	660	1.37	600	1.26
20	960	1.49	900	1.38	830	1.28	760	1.18	690	1.07
30	1 190	1.24	1 100	1.14	1 010	1.04	920	0.94	820	0.86
40	1 380	1.10	1 270	1.01	1 150	0.91	1 040	0.82	920	0.74
50	1 550	1.00	1 440	0.91	1 330	0.82	1 170	0.74	1 010	0.65
60	1 710	0.93	1 560	0.85	1 410	0.76	1 260	0.68	1 100	0.60
90	2 090	0.78	1 900	0.71	1 700	0.64	1 510	0.56	1 310	0.49
120	2 420	0.70	2 190	0.64	1 960	0.57	1 730	0.50	1 500	0.43

Tables 10, 11, 12, and 13 have been prepared by the use of the foregoing formulas, all based on

$$I = \frac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$$

$$i = 5$$

and an absorption of water by the soil at the rate of about $\frac{3}{8}$ in. per hour at the beginning of a critical period, decreasing gradually to a rate of $\frac{1}{3}$ in. per hour at the end of 1 hour.

In the case of conduits already constructed, neither the value of $C_T - C_B$, nor the value of d_m, taken from the tables for $t_m = t_D$, should be considered final.

To facilitate the use of the formula and of the reduction factors, a, Table 13 has been prepared. In this table the value of

$$\frac{0.71}{\dfrac{2\,t_m}{t_m+60} + t_m^{\,0.4}}$$

appears in Column 3, and the value of

$$\left[27.5\sqrt{t_m}-(G_T-G_{T+5})\right]_{\substack{a=1\\b=1}}$$

in Column 2. The value of $a\ b$ is given in Columns 4 to 18.

To find the discharge, by this table, multiply the value found in Column 3 by the appropriate factor taken from Columns 4 to 18; and to find the increase of the water stored in the conduits, multiply the values found in Column 2 by the same factor.

The table, as already stated, is based on the assumption that $i = 5$; and that, during the time, t_m, the absorption of water by exposed soil surface is at rates decreasing gradually from about $\frac{3}{3}$ in. per hour to $\frac{1}{3}$ in. per hour at the end of 1 hour.

TABLE 12.—STORM-WATER FLOW.

FOR $b = 12$, IN THE FORMULA, $I = \dfrac{b}{\dfrac{2\ t}{t+60}+t^{0.4}}$

All values in this table are based on

$$d_m = \frac{0.71\ a\ b}{\dfrac{2\ t_m}{t_m+60}+t_m^{\,0.4}} = \frac{4.36\ a\ R}{\dfrac{2\ t_m}{t_m+60}+t_m^{\,0.4}}$$

$$S_m = 27.5\ a\ b\sqrt{t_m}$$

and $C_T - C_B = S_m - (G_T - G_{T+5})$

$$b = 12 \text{ and } R = 1.95.$$

Duration of critical period, t_m, in minutes.	ALL IMPERVIOUS.		75% IMPERVIOUS.		50% IMPERVIOUS.		25% IMPERVIOUS.		NONE IMPERVIOUS.	
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.
5	100	4.15	100	4.03	95	3.90	90	3.78	90	3.65
10	980	3.05	900	2.95	860	2.84	820	2.72	780	2.60
15	1 210	2.54	1 160	2.44	1 110	2.34	1 060	2.21	1 010	2.11
20	1 440	2.23	1 380	2.14	1 310	2.03	1 240	1.92	1 170	1.80
30	1 790	1.86	1 700	1.76	1 610	1.68	1 520	1.58	1 420	1.47
40	2 070	1.66	1 960	1.57	1 840	1.48	1 730	1.39	1 610	1.29
50	2 330	1.50	2 200	1.41	2 060	1.32	1 930	1.25	1 790	1.15
60	2 560	1.39	2 410	1.31	2 260	1.22	2 110	1.14	1 950	1.06
90	3 140	1.18	2 950	1.11	2 750	1.04	2 550	0.95	2 350	0.88
120	3 620	1.04	3 390	0.98	3 150	0.90	2 910	0.83	2 670	0.77

TABLE 13.—VALUES OF $a\,b$,

WHEN $I = \dfrac{b}{\dfrac{2\,t}{t+60} + t^{0.4}}$ AND $d_m = a\,b\,\dfrac{0.71}{\dfrac{2\,t_m}{t_m+60} + t_m^{\,0.4}}$

$S_m = a\,b\,(27.5\,\sqrt{t_m})$ and $C_T - C_B = a\,b\,[C_T - C_B]_{\substack{a=1\\ b=1}}$.

Duration of critical period, t_m, in minutes.	Base Values, for $b=1$ and $a=1$.		Values of $a\,b$:															
			For $b=4$ and $R=0.65$.					For $b=8$ and $R=1.30$.					For $b=12$ and $R=1.95$.					
	$C_T - C_B$ cubic feet per acre.	d_m second-feet per acre.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.	All impervious.	75% impervious.	50% impervious.	25% impervious.	None impervious.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
5	8	0.347	4.00	3.65	3.30	2.90	2.60	8.00	7.70	7.30	6.95	6.55	12.00	11.60	11.20	10.80	10.60
10	78	0.254	4.00	3.55	3.10	2.65	2.15	8.00	7.50	7.05	6.65	6.15	12.00	11.55	11.10	10.70	10.20
15	107	0.212	4.00	3.50	2.95	2.40	1.90	8.00	7.45	6.95	6.50	5.90	12.00	11.50	10.95	10.45	9.96
20	125	0.186	4.00	3.45	2.90	2.25	1.70	8.00	7.40	6.90	6.30	5.75	12.00	11.40	10.90	10.30	9.70
30	150	0.156	4.00	3.40	2.75	2.10	1.50	8.00	7.35	6.75	6.10	5.50	12.00	11.40	10.80	10.20	9.50
40	170	0.138	4.00	3.35	2.70	2.00	1.35	8.00	7.35	6.65	6.00	5.30	12.00	11.35	10.70	10.10	9.35
50	195	0.125	4.00	3.30	2.65	1.95	1.25	8.00	7.30	6.60	5.90	5.20	12.00	11.30	10.60	9.95	9.20
60	215	0.116	4.00	3.30	2.60	1.90	1.15	8.00	7.30	6.55	5.80	5.10	12.00	11.30	10.55	9.85	9.10
90	260	0.098	4.00	3.25	2.50	1.75	1.00	8.00	7.25	6.50	5.75	5.00	12.00	11.30	10.50	9.70	9.00
120	300	0.087	4.00	3.25	2.50	1.70	0.90	8.00	7.25	6.50	5.75	4.95	12.00	11.30	10.50	9.69	8.90

The type of the formula for rain intensity, as here recommended, is the direct outcome of an attempt to make the resulting values follow closely the San Francisco limiting rain curve for all values of t up to 24 hours.

The parabola, as a limiting curve, was tried, but, for higher values of t, departed too far from the curve of 1899 to be considered entirely satisfactory.

When all time periods that come into consideration are less than 2 hours, as will be the case in nearly all urban problems, a parabola can frequently be found which will closely approximate the limiting rain curve, and may be used in its stead.

In such case let the equation of the limiting rain curve be

$$q = e\sqrt{t} \text{ (total rainfall in inches)} \quad \ldots\ldots (41)$$

The equation of the surface run-off curve will then be

$$q' = a\, e\sqrt{t} \quad \ldots\ldots (42)$$

The value of a, as has already been explained, is dependent upon the amount of impervious surface in any area, and varies, too, with the duration of the critical period. For impervious areas, a is unity.

For $t = 60$, the value of q will be the maximum rainfall in 1 hour, therefore, from Equation 41,

$$R = e\sqrt{60} \quad \ldots\ldots (43)$$

$$e = \frac{R}{7.746} \quad \ldots\ldots (44)$$

$$e = 0.129\, R \quad \ldots\ldots (45)$$

$$\text{therefore } q' = 0.129\, a\, R\sqrt{t} \quad \ldots\ldots (46)$$

The maximum intensity of the rainfall during any number of minutes can be found from

$$I = \frac{60\, q}{t} = \frac{60\, e}{\sqrt{t}} \quad \ldots\ldots (47)$$

$$\text{and } I = \frac{7.75\, R}{\sqrt{t}} \quad \ldots\ldots (48)$$

By reference to Fig. 10 it can be seen that for a curve the equation for which is

$$q' = a\, e\sqrt{t} \quad \ldots\ldots (42)$$

the equation of the line, $B'\ T$, can be written

$$y = \frac{0.414\, a\, e}{\sqrt{t}}\, x + 0.586\, a\, e\sqrt{t} \quad \ldots\ldots (49)$$

For $x = \frac{2\, t}{3}$, this equation will give the ordinate of the point, E

The locus of the point, E, will be expressed by

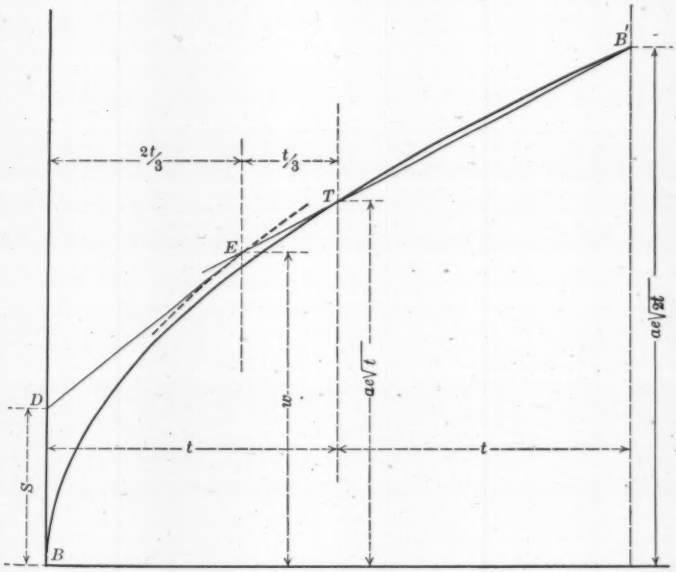
$$w = \frac{0.828\ a\ e\ t}{3\sqrt{t}} + 0.586\ a\ e\sqrt{t} \quad \ldots\ldots(50)$$

$$w = 0.862\ a\ e\sqrt{t} \quad \ldots\ldots(51)$$

$$\text{or } w = 1.056\ a\ e\sqrt{x} \quad \ldots\ldots(52)$$

Therefore the tangent to the E-curve at the point, E,

$$\frac{d\,w}{d\,x} = \frac{0.528\ a\ e}{\sqrt{x}} = \frac{0.647\ a\ e}{\sqrt{t}} \quad \ldots\ldots(53)$$



WHEN THE LIMITING RAIN CURVE IS A PARABOLA

FIG. 10.

The value of $\frac{d\,w}{d\,x}$ as thus found will be inches in depth in 1 min. Therefore d_m, in inches in 1 hour, or in second-feet per acre,

$$d_m = 60\,\frac{0.647\ a\ e}{\sqrt{t}} = \frac{38.8\ a\ e}{\sqrt{t_m}} \quad \ldots\ldots(54)$$

And, by substitution,

$$d_m = \frac{5.00\ a\ R}{\sqrt{t_m}} \text{ cu. ft. per sec. per acre} \quad \ldots\ldots(55)$$

The value of S, expressed in inches of depth, will be

$$w - \frac{2\,t}{3}\left(\frac{0.647\,a\,e}{\sqrt{t}}\right)$$

from which it follows (writing t_m for t, and S_m for S):

$$S_m = 3\,630\,[0.862\,a\,e\,\sqrt{t_m} - 0.431\,a\,e\,\sqrt{t_m}] \quad \ldots\ldots (56)$$

$$S_m = 1\,565\,a\,e\,\sqrt{t_m} \text{ cu. ft. per acre} \quad \ldots\ldots (57)$$

From Equation 57, $$t_m = \left(\frac{S_m}{1\,565\,a\,e}\right)^2 \quad \ldots\ldots (58)$$

By substitution from Equation 45,

$$t_m = \left(\frac{S_m}{202\,a\,R}\right)^2 \quad \ldots\ldots (59)$$

and $$S_m = 202\,a\,R\,\sqrt{t_m} \quad \ldots\ldots (60)$$

Finally, from Equations 54 and 58,

$$d_m = \frac{60\,700\,a\,e^2}{S_m} \quad \ldots\ldots (61)$$

and from Equations 55 and 60,

$$d_m = \frac{S_m}{40.4\,t_m} \quad \ldots\ldots (62)$$

The value of a, for use in the foregoing equations, may be taken from Table 6, for the soil absorption conditions on which that table is based.

For San Francisco, the value of e to be used in these equations is 0.0775, the value of R, the greatest possible rainfall in 1 hour, being 0.60 in.

For the rainfall conditions prevailing on the Atlantic Slope, the value of e will ordinarily lie between 0.15 and 0.25.

In order that the importance of using good judgment in the construction of the limiting rain curve may become apparent, Tables 14 and 15 have been prepared.

In Table 14 corresponding values of S_m and d_m are given for the various values of t_m, estimated in the one case by the use of formulas based on limiting rain curves of the San Francisco type, and in the other by the use of formulas based on the parabola type of curve. This table is of general application.

In Table 15 the comparison is made between corresponding values of t_m and d_m for selected values of S_m; but in this case a specific value of R was assumed: $R = 0.60$. The figures on the same lines are not those that would result from an adaptation of curves of the two

types to the rain records, because properly selected curves may not have coincident values of R.

For the San Francisco type of curves:

$$I = \frac{b}{\frac{2\,t}{t+60} + t^{0.4}}$$

$$d_m = \frac{4.36\,a\,R}{\frac{2\,t_m}{t_m+60} + t_m^{0.4}}$$

and $$S_m = 169\,a\,R\sqrt{t_m}$$

For the parabola:

$$I = \frac{60\,e}{\sqrt{t}}$$

$$d_m = \frac{5\,a\,R}{\sqrt{t_m}} = \frac{S_m}{40.4\,t_m}$$

$$S_m = 202\,a\,R\sqrt{t_m}$$

TABLE 14.—COMPARATIVE VALUES OF STORM-WATER FLOW ESTIMATED FOR TWO TYPES OF THE LIMITING RAIN CURVE.

For impervious areas throughout.

$a = 1$

Duration of the critical period, t_m, in minutes.	FOR A LIMITING RAIN CURVE OF THE SAN FRANCISCO TYPE.		WHEN THE LIMITING RAIN CURVE IS A PARABOLA.	
	S_m, cubic feet per acre.	d_m, second-feet per acre.	S_m, cubic feet per acre.	d_m, second-feet per acre.
5	378 R	2.13 R	452 R	2.24 R
10	535 R	1.56 R	640 R	1.58 R
15	655 R	1.30 R	780 R	1.29 R
20	755 R	1.14 R	905 R	1.12 R
30	930 R	0.96 R	1 010 R	0.91 R
40	1 070 R	0.85 R	1 280 R	0.79 R
50	1 200 R	0.77 R	1 430 R	0.71 R
60	1 310 R	0.71 R	1 570 R	0.65 R
90	1 600 R	0.60 R	1 920 R	0.53 R
120	1 850 R	0.54 R	2 220 R	0.46 R

Application of the New Method to Special Cases.—Should it be desired to know the extent to which a branch conduit contributes to the discharge of a main conduit at the time, T, two procedures are possible. The discharge estimate may be made for the main conduit for points just above and just below the entrance of the sub-main. The difference between the two results will be the required discharge of the sub-main.

TABLE 15.—Comparative Values of Storm-Water Flow Estimated for Two Types of the Limiting Rain Curve.

For $R = 0.60$.

For impervious areas throughout.

$a = 1$, and $R = 0.60$.

Increase of storage, S_m, in cubic feet per acre.	For a Limiting Rain Curve of the San Francisco Type.		When the Limiting Rain Curve is a Parabola.	
	t_m, minutes.	d_m, second-feet per acre.	t_m, minutes.	d_m, second-feet per acre.
200	3.89	1.43	2.72	1.82
250	6.08	1.17	4.25	1.46
300	8.75	0.99	6.11	1.21
400	15.6	0.77	10.9	0.91
500	24.2	0.64	16.9	0.73
600	35.0	0.54	24.4	0.61
700	47.5	0.47	33.4	0.52
800	62.0	0.42	43.6	0.45
900	78.6	0.38	55.0	0.40
1 000	97.3	0.35	68.1	0.36

Or, as may frequently be convenient, the sub-main is considered separately. In this event it must be remembered that the mass curve of surface run-off, which is of most unfavorable shape for the main district, is as shown in Fig. 8. During the first part of the critical period, the rainfall will have its greatest intensity. The run-off produced by the rain of greatest intensity will have time to escape from the sub-district (supposed to be relatively small) before the main conduit flows at capacity. Therefore, in applying the graphical method, only that part of the run-off curve is to be used which lies nearest to T. But, for decreasing intensities of rain, in a small district, there will probably be less water in the conduits at the time, T, than t_D minutes before T (t_D now applying to the sub-district). Consequently, instead of an augmentation of water quantity stored in the conduits there will be a depletion of this storage. The value of $C_T - C_B$ or of S, as the case may be, will be negative, and must be platted upward from the curve at T, instead of downward.

In the supposed case, Fig. 8, the sub-district time is 20 min. and the required discharge of the sub-main is represented by the inclination of the line, $E_1 D_1$.

It may be of interest to note that the described method of estimating flow at any point of a water conduit when the rates are known at which

water is supplied to it by its feeders, is applicable to rivers and canals* as well as to covered conduits, and also to conduits operating in part under pressure.

A closed conduit serving throughout under pressure is an extreme case. There can then be no augmentation of the quantity of water stored in the conduit. The effect of an increase or decrease in the rate of supply will be instantaneous from end to end of the conduit. The time, t, will become zero. The points, B, T, and E on the curve will all be at the origin of the co-ordinates, and the maximum rate of flow at all points of the conduit will be equal to the maximum rate at which water is supplied, because, for $t = 0$, the E-curve is tangent to the supply curve at the point, B.

At the other extreme is the case of flow through large reservoirs or basins. In this case, the storage of water is large, the time, t, is relatively long, and the maximum rate of outflow is correspondingly small.

In applying the method of estimating required storm-water capacity, as described, it is well to remember that a proper determination of the value of b eliminates the necessity for introducing into the calculation any further allowance for safety margin except such as may be covered by a selection of a value for the factor a. As a is unity for areas with impervious surfaces throughout and as the value of a is otherwise definitely determinable, it will be seen that the room for error is small.

General Deductions.—It appears from the foregoing:

1.—That the method as described of estimating storm-water flow is applicable to districts of any shape, size, or surface condition.

2.—That if a conduit affords no reservoir space, the storm-water flow at all points thereof would be equal to the rate of inflow at the storm-water inlets, and the effect of an increase or decrease of this rate would be immediately manifest at all points of the conduit. (This is the case of a conduit under pressure.)

3.—That the longer the time required for storm-water concentration, the larger will be the unit storage capacity of the conduits in any district, or *vice versa*, and the greater the influence of the conduit storage upon the rate of flow.

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* This is substantially the method of estimating river flow described by the writer in a recent discussion of a paper on a California flood. *Transactions*, Am. Soc. C. E., Vol. LXI, p. 332.

4.—That an outfall conduit, receiving no further accessions, below a certain point:

a.—If occasionally or always submerged must be given uniform capacity from that point to the point of outfall;

b.—If not flowing under pressure, and if it have a free outfall not obstructed by back-water, its maximum discharge will decrease somewhat with increasing distance below that point.

If the shape of the limiting rain curve has been properly determined from adequate rainfall observations, the secondary curves derived therefrom should be smooth curves. The value of discharge determined therefrom will, in other words, follow a definite law of change.

It is obvious that the graphical method of determining discharge may be applied to any rain storm for which a mass curve of the rainfall can be platted. The proper location of the points, B and T, on the resulting surface run-off curve will be found by trial.

The method of estimating storm-water flow as here described commends itself for use whenever the limiting rain curve is known:

First.—Because the maximum fall of rain in all possible time intervals is taken into account.

Second.—Because the element of uncertainty relating to the rate of rain-water inflow into storm-water inlets is confined within narrow limits, and the possible error in the selection of a reduction factor is correspondingly small.

Third.—Because the essential element, the increase of water storage in the conduit system during a critical period of time, is subject to determination with reasonable accuracy.

Fourth.—Because it is simple and generally applicable.

Storm-Water Relief Outlets and Sewer Capacities.

The ordinary principle of providing storm-water relief outlets from the sewers of the combined system, where opportunity offered, has been adhered to in designing the storm-water conduits for San Francisco. The infrequency of rain in excess of ½ in. per day made it appear desirable to let the relief outlets come into service only when the rainfall rate exceeded this amount. In other words, the sewers were planned to carry the run-off due to a rain of ½ in. in 24 hours to the sewer outfall. The remainder of the flow during storms will pass through the relief outlets on the shortest course to bay or ocean.

The sewers of the combined system, therefore, were given a capacity above relief outlets to carry:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall of greatest intensity;

Some allowance as a safety margin.

Below relief outlets:

The sewage proper;

A certain amount of soil-water, entering by leakage;

The run-off due to a rainfall at the rate of ¼ in. in 24 hours.

The sewer capacity thus determined for points below relief outlets is nearly four times as great as the mean flow of sewage proper. The storm-water reliefs, therefore, should not come into service with more than 25% of sewage in their outflow, and this amount only at times when their outflow is small. When in service at full capacity the sewage carried by them will be so dilute as to be quite inoffensive.

Sewage Disposal.

Dilution and Outfall Points.

Consideration was given to the various methods of sewage disposal which have been developed and found satisfactory.

Utilization by application to land was entirely out of the question, owing to San Francisco's location at the extreme point of a narrow peninsula, and the remoteness of lands which would be suitable for cultivation under sewage irrigation.

Treatment in filter beds, to produce an inoffensive effluent, or chemical treatment for like purpose, would entail much additional cost and is not at present necessary for any part of San Francisco's sewage, owing to the great dilution that can be secured, and the favorable set of the currents from the selected outfall points away from the shore.

Dilution in the waters of the bay and the ocean is without doubt for San Francisco the natural and proper method of sewage disposal.

The questions to be considered, therefore, relate to the best points of outfall and the best method of conveying the sewage to those points.

Without discussing at length the studies of tidal currents made in 1893 with floats, and the results of current measurements made from anchored boats by the United States Coast and Geodetic Survey some years before, the conclusions reached will be briefly stated.

The float observations did not cover the entire water front, nor all possible stages of the tide. The means were not at command to accomplish this. Generally known facts or opinions relating to the direction and velocity of the bay currents led to the selection of certain points from which the floats were started at various stages of the tide and then followed day and night by boat until stranded or well started seaward. The float was of wood about 6 ft. long, weighted at one end and carrying a flag or lantern at the other.

Float observations were made from points off Hunter's Point, Potrero Point, Center Street Pier, and Powell Street Pier. At each of these points floats were started at low tide, at high tide, and at intermediate stages. There were no float observations made oceanward from Golden Gate as these did not seem necessary. The current in and out is swift. Someone has computed that 1 cu. mile of water passes through Golden Gate each way every day. This vast volume, as it flows outward, entrains to some extent the water that lies off the beach, southward from Point Lobos. This is probably the reason why the ocean beach is exceptionally free from drift.

All the Hunter's Point flood-tide floats passed up the bay, clear of tide marsh and shore line. The ebb-tide floats cleared Potrero Point by more than half a mile and passed east of Mission Rock.

Floats started at flood tide from the Sugar Refinery wharf at Potrero Point took a course inside of a line drawn from Potrero Point to Hunter's Point; one was caught and held in the flood-tide eddy south of Potrero Point. At ebb tide all floats passed to the east of Mission Rock into the deep water of the bay.

The floats started at flood tide from the wharf of the Pacific Rolling Mill, farther north on Potrero Point, all hugged the shore toward Hunter's Point. The ebb-tide floats showed equally unfavorable conditions. Their course was close to the end of Center Street pier, to the westward of Mission Rock.

Floats started at flood tides from the end of Center Street pier, oil wharf, passed close to Potrero Point. Those at ebb tide moved toward Mission Rock.

The floats started at the end of Powell Street pier on flood tides in part took a course somewhat off shore into deep water, in part they went gradually shoreward. The ebb-tide floats all went off shore more or less, most of them clearing Black Point by more than one-quarter mile.

The conclusion reached from the foregoing observations was that sewage might safely be discharged into the bay or ocean on the northern frontage of the city, provided only that the outfall be arranged far enough off shore in deep water.

The eastern frontage of the city, from near Market Street to near Hunter's Point, is not a desirable locality for disposing of sewage. It was recognized, however, that the discharge of sewage in limited amounts through submerged outlets in deep tidal waters is rarely objectionable, and, therefore, that small shore areas which cannot at once be made tributary to the contemplated main sewers, may, with proper restrictions, be allowed to discharge into the bay.

Hunter's Point was found to offer exceptional advantages for a delivery of sewage into swift tidal currents with ample and thorough dilution.

The following points of outfall were finally selected:

Off North Point, with discharge in at least 36 ft. of water at 1 200 ft. or more from the water-front line.

Off Hunter's Point, in a corresponding depth.

Off the ocean shore just west of Baker's Beach, at the foot of 27th Avenue.

Off the foot of Scott Street, in at least 36 ft. of water, and well off shore.

Two other points were selected, though not to be utilized at once. These are Fort Point, where it may be found desirable after many years to concentrate all sewage reaching the northern city front, and Point Lobos, where it was foreseen that the sewage of the Sunset and other ocean-slope districts should ultimately be delivered.

Of the areas tributary to the selected outfall points it may be said:

The North Point main is to receive the sewage from an area of about 11 500 acres, which includes the greater portion of the residence district of the city and the entire business section. About 80% of the city's population was within this area at the time the report was written in 1899.

The territory tributary to the Hunter's Point main comprises 2 300 acres.

The Point Lobos outfall will ultimately receive the sewage from the greater portion of the ocean slope of the city.

The Scott Street outfall will serve the Harbor View district, until it becomes desirable to extend the North Point main to Fort Point.

The conclusion having been reached that the best points for the discharge of sewage are the projecting points of the water front which are swept by swift tidal currents, it became necessary to give Fort Point special consideration. Fort Point, at the south side of Golden Gate, is without doubt the best location for an outfall, but, for the present, it is inaccessible, owing to remoteness from the built-up section of the city, and the great cost that would be involved in reaching it in advance of general water-front improvement.

In the order of their desirability as points of outfall for the sewage of bay-slope districts there seems to be no question that Fort Point should be ranked first, though, as explained, not now available; North Point, second, and Hunter's Point, third.

Consequently, the collecting system of sewers has been planned so as to make the greatest area possible tributary to the North Point outfall, and let Hunter's Point take care of the remainder.

This is the fundamental reason why, as elsewhere explained, the upper portion of the Islais Creek drainage is to be cut off from the Hunter's Point system and made tributary by gravity flow to the North Point main, by tunneling along San José Avenue under College Hill. By this arrangement, moreover, an immediate installation of the outfall sewers for an important outlying district becomes possible, and it has the further advantage of reducing the amount of sewage to be pumped over from the Hunter's Point main into the North Point sewer system if, in the future, it ever becomes necessary to abandon the Hunter's Point outfall. Furthermore, it appears unobjectionable to permit the temporary delivery of sewage in small amounts at other points of the bay front, provided the discharge be always effected well off shore in deep water.

Sewerage Districts.

Sewerage districts were thereupon laid out in conformity with these principles, and the plans for the systems of collecting conduits were completed.

The areas noted in Table 16 and the estimated population, which is really the assumed distribution of population when San Francisco has reached the 1 000 000 mark, were used as guides in estimating, by methods already described, the quantity of sewage, soil-water, and storm-water to be cared for in the sewers.

TABLE 16.—SAN FRANCISCO SEWER DISTRICTS.

Name of district and sub-district.	Area, in acres.	Estimated future population.
North Point District:		
County Line West	836	24 800
County Line East	381	8 500
Persia Avenue	246	7 300
Sunnyside	579	21 700
Glen Park	582	17 500
Silver Avenue West	177	5 300
Valley Street and Upper Army Street	794	31 700
West Bernal Heights	149	6 000
Lower Islais Creek	1 125	47 900
West Potrero, Upper Twenty-second Street	620	31 000
Eighteenth Street	660	33 000
Fourteenth Street	1 307	74 500
Miscellaneous	27	1 600
Eleventh Street	978	54 400
East Potrero	334	16 700
Fifth Street, Sixth Street, Seventh Street and Miscellaneous	782	103 700
Kearny Street	41	6 200
Mission Flats	845	72 500
North Rincon Hill	126	15 700
Portsmouth	157	31 000
Yerba Buena	279	33 500
North Beach	500	30 000
Totals	11 525	674 500
Hunter's Point District:		
College Homestead	122	3 700
Gaven Street	68	2 000
St. Mary's	91	3 600
South Bernal Heights	123	4 900
University Mound	488	14 600
West Silver Terrace and Miscellaneous	65	2 000
North Silver Heights	77	3 100
Railroad Avenue	115	4 600
North Slope, Hunter's Point	306	12 200
Bay View	590	23 600
South Slope, Hunter's Point	249	10 000
Totals	2 294	86 300
Harbor View District	816	48 100
Richmond	1 200	60 000
West Richmond	212	19 200
Upper Sunset	1 658	64 200
Lower Sunset	2 564	98 600
Ocean View	783	29 800

POPULATION, WATER CONSUMPTION, AND SEWAGE.

Before describing the district systems, a word is to be said relating to the prospective growth of San Francisco, and the water consumption. The population estimate, as made in 1899, is shown by the diagram, Fig. 11. For comparison, the population of several other cities of the United States is also shown. The conclusion was reached that about 50 years would elapse before San Francisco would have a population of 1 000 000. Fifty years being a reasonable time for which to insure adequate service to all parts of the city, and fully realizing that the

forecast now made for a period so remote is to be regarded other than as a probability, this population was accepted as that on which estimates of water consumption and, therefore, volume of sewage proper and the like, should be based.

The information presented in Fig. 11, and extended to include the results of the census of 1900, is shown in Table 17.

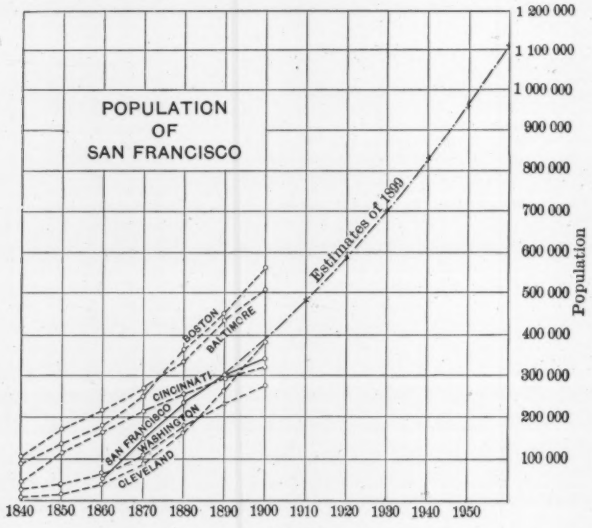


FIG. 11.

TABLE 17.—POPULATION OF CITIES FROM UNITED STATES CENSUS, AND ESTIMATED.

Year.	Washington.	Cleveland.	Cincinnati.	San Francisco.	Baltimore.	Boston.
1840	23 364	6 071	46 338	.....	102 313	93 383
1850	40 001	17 034	115 435	.....	169 054	136 881
1860	61 122	43 417	161 044	56 802	212 418	177 840
1870	109 199	92 829	216 239	149 473	267 354	250 526
1880	177 624	160 146	255 139	233 950	332 313	362 839
1890	230 392	261 383	296 908	298 997	434 439	448 477
1900	278 718	381 768	325 902	342 782	508 957	560 892
1910	.....	.....	.....	*480 000	.....	.....
1920	.....	.....	.....	*590 000	.....	.....
1930	.....	.....	.....	*700 000	.....	.....
1940	.....	.....	.....	*830 000	.....	.....
1950	.....	.....	.....	*960 000	.....	.....
1960	.....	.....	.....	*1 100 000	.....	.....

* Estimates made in 1899.

As a result of the destruction of the business section and a large part of the residence district of San Francisco by fire in 1906, there was at that time a sudden drop in the actual population curve.

If recast at the present time there would, no doubt, be some departure from these estimates.

From information available in 1899, it was found that, exclusive of water pumped from bay or ocean for the supply of bathing tanks, for use in street sprinkling, and for use in condensers, about 70 gal. per capita per day were supplied to the city by the Spring Valley Water Company. The maximum daily per capita consumption at that time was 82 gal.

It is believed that San Francisco's cool climate, particularly in the summer, the compact development of the residence districts, with restricted lawn and garden areas, and the natural difficulties in the way of bringing in a copious supply, will always compel more than ordinary economy in the use of water. It is not likely, therefore, that the present consumption of 75 gal. per day will ever be greatly exceeded. Allowing 20% as the excess of the maximum daily consumption over the mean, and making a small allowance for water taken from ocean, bay and private wells (4% was assumed), the conclusion was reached that 94 gal. per day should be taken into account in estimating the volume of sewage proper.

About 75% of the water used for all purposes in the city will reach the sewers. But the rate at which the water is used and delivered to the sewers is not uniform. It was assumed, without special local studies to confirm the accuracy of this assumption, that the maximum flow would exceed the mean by 50%.

Based on these premises, it was estimated that for each 1 000 inhabitants the average quantity of sewage proper, on days of maximum water consumption, would be 0.109 sec-ft., and that the maximum rate at which this would reach and be carried off by the sewers would be 0.163 sec-ft. This was the unit adopted for use in estimating the required capacity.

Soil-Water.

The quantity of soil-water entering the sewers by leakage depends on the care with which the sewers are constructed, their permeability, leaky joints, and also the relative elevation of the water-table and sewer. When the water-table is below the sewer there may be leakage from the sewer into the ground, but there will be no reverse flow.

Throughout a large portion of San Francisco the sewers will lie above the soil-water plane. If winter rains bring the water-table up to cover the sewer, in the high parts of the city, it will be for short periods only. Therefore, very little inflow from the soil into the sewers is to be assumed, except in the low marginal and down-town areas, where the plane of saturation lies above the sewers. It would be interesting to know whether this plane could not be permanently lowered, in view of the fact that the bay front of the city is, or was, skirted by mud flats which are to be considered nearly impervious. But this cannot be ascertained short of very expensive experiments.

If there is a great leakage into the sewers, they may have some effect on the water-table, and the leakage will gradually decrease until equal to the ground-water replenishment, from whatever source this may come. In the absence of adequate information relating to ground-water depth in different parts of the city, a uniform allowance for an increment of the flow in the sewers due to this cause was made for all the high portions of the city and a larger allowance for the low areas. The ground-water reaching the sewers from high ground was introduced into the calculation of sewer capacity at 0.001 sec-ft. per acre, and that from the low parts of the city, where the sewers, for the most part, will lie below the natural soil-water plane, at 0.003 sec-ft. per acre.

The Tides of San Francisco Bay.

The range of tides in San Francisco Bay is thus referred to in the *Pacific Coast Pilot,* by George Davidson, Hon. M. Am. Soc. C. E., who for many years was in charge of the work of the United States Coast and Geodetic Survey on the Pacific Coast:

"From the lower low water ('low water large') the tide rises for about 7½ hours, say 4.4 ft. to the smaller of the two high tides ('high water small'), then falls 1.4 ft. in less than 4½ hours to the 'low water small,' which is higher than the preceding low water; then rises say 2.9 ft. in 6¼ hours to the higher high water or 'high water large;' it then falls again 5.8 ft. in over 7 hours to the lower low water, or 'low water large.'

"Instead of the above figures, the fall from high water small, or 'half tide' to the 'low water small,' may range from 3½ ft. at one position of the moon to 0.3 ft. at another; in the latter case there will be apparently a long stand of about 5 hours.

* * * * * * * * * *

"The average difference of the higher high and lower low waters of the same day is 5.2 ft., with a greatest observed range, noted for February 8th, 1876, of 9.93 ft."

The "Tide Tables" for 1899 of the United States Coast and Geodetic Survey gave the following information:

The expected mean range of tides is 4.5 ft.
The expected mean range of spring tides is 5.4 ft.
The expected mean range of neap tides is 3.5 ft.
The expected mean range of great tropic tides is 7.2 ft.
The expected maximum range of tides in 24 hr. is 7.8 ft.
The expected minimum range of tides in 24 hr. is 3.1 ft.

From these tidal ranges, and the area of the bay, together with the area and elevation of the adjoining submersible land, and with proper allowance for the difference in the time at which the high and low tides occur, the total volume of flow through the Golden Gate can be estimated.

To the outflow through the Golden Gate, however, there is to be added the flow of the rivers tributary to the bay. This is not inconsiderable.

It is the run-off from more than 60 000 sq. miles of tributary area. The two rivers, the Sacramento and the San Joaquin, contribute more than 4 000 sec-ft. at their lowest autumn stage, and more than 80 000 sec-ft. throughout the first six months of the year, often reaching from 150 000 to 200 000 sec-ft. for short periods of time.

River flow through the bay alone would effect sewage dilution from 30 to 1 500 fold, yet the river water is only a trifle when compared with the great return flow of waters which have entered the Golden Gate on a rising tide.

North Point Sewer District.

The North Point Sewer District has an area of 11 525 acres. Its estimated population, when that of the city, of present area, is 1 000 000, will be 674 500.

About 80% of the city's population in 1899 lived within this district. It includes the northern frontage of the city easterly from Black Point, the eastern frontage as far south as the hills on the south side of Islais Creek, and extends inland to the summit of the peninsula ridge and along the same, south to, and slightly beyond, the county line, including all the Islais Creek water-shed already described.

Nearly all this district is to be sewered on the combined system. The old sewers are utilized to the greatest extent possible.

The position of the main intersecting sewer was determined by two governing points, the outfall at North Point and the point on the old line of Mission Creek where the sewage of the Mission is to be intercepted. The sewer was placed low enough to serve the district near Harrison and Fourteenth Streets. At that point many street grades have been established without due regard to future requirements, and to the permanent injury of adjoining property. Improvements of street surface and improvements on private land have been made to conform to these grades, many of which are too low. It is doubtful whether material correction of such grades will ever be possible. The North Point main, therefore, was planned to fit the grades as established, the aim having been to keep the hydraulic grade line at least 1 or 2 ft. below the street surface at the critical points. This was the best that could be done for this district which, under present conditions, is submerged whenever there is a heavy downpour of rain. In order that the reason for small safety margin, for the scant elevation at a few points of street surface above the hydraulic gradient of the full conduit, may be better understood, it may be stated that there were such points, as at Eighteenth and Division Streets, where the grades marked out a pot-hole arrangement. The official grade there at the time the sewer system was planned was 7.5 ft. above city base,* while at all street intersections around this point the official grades ranged from 10.5 to 13 ft. At Thirteenth and Harrison Streets there is a similar arrangement. Street grades of 6 and 6.5 ft. are entirely surrounded by official grades several feet higher. Official street elevation is similarly defective at the northern termination of York, Florida and Alabama Streets, and at the intersection of Fifteenth and Shotwell Streets; also at Sixteenth and Shotwell Streets; at the eastern termination of Fifteenth Street, on Folsom Street midway between Sixteenth and Seventeenth Streets, and at Fourteenth Street; and there are still other points where grades should be changed so as to eliminate completely the pot-hole feature.

In the past this district has suffered one inundation after another. The proposed increased capacities of the sewers and storm-water drains will greatly ameliorate this condition, but the mischief done by the

* The grade at this point was raised a few years after the report was written.

establishment of faulty street grades cannot be entirely undone by the construction of sewers, and it must be left to the property owners to find out how much better off they would be if the high-water line in the sewers could be kept 5 or 6 ft. below the street surface instead of rising almost to it. The only satisfactory relief from the local floods that may result from defective or choked storm-water inlets and from the annoyance of water in cellars, or, at least, of interrupted cellar drainage during severe rain storms, will result from raising the street grades, particularly at the points where pot-hole features are to be eliminated. The present property owners are often the innocent sufferers, from the desire of their predecessors to have official grades established at natural ground height or but little above it, in order to save the cost of grading.

The head of the North Point main, if considered as an interceptor, is far out on the Mission Road, at the point, in fact, where this road crosses Islais Creek. From this point the sewer will be in tunnel for a distance of 4 500 ft. following the line of Mission Road and San José Avenue.

At Army and Valencia Streets the sewage now flowing down Army Street sewer is to be intercepted and diverted northward. The present sewer eastward from this point will continue in service as a storm-water relief outlet, and as a main from which a mile farther east sewage will be pumped to the nearest point of the North Point main.

The course of the North Point main is almost due north following the streets best adapted by reason of elevation to receive it. It will be on Twenty-sixth Street from Valencia Street to Treat Avenue, on Alabama Street from Twenty-fifth to Nineteenth Streets, and will reach Division Street at Eighteenth Street. It will follow Division Street to Harrison Street, paralleling the present Channel Street sewer; thence it will be on Harrison Street to Eighth, on Eighth to Howard, thence on Howard and New Montgomery or Second Street northwest to Market Street and north to the water front on either Montgomery or Sansome Street. All along its course it will intercept the sewers entering it from the west.

At Eighteenth and Division Streets, at Fifteenth Street, and about on the line of Fourteenth Street, will be storm-water relief outlets of large capacity. The outlet arrangement at Eighteenth Street is practically that of two parallel conduits with a common roof, separated from

each other by a dividing overfall weir. The entire flow will be confined by this weir to the North Point main until the full capacity is reached, whereupon the excess flow will go over the weir into the storm-water conduit.

Other storm-water outlets will be provided at Seventeenth, Sixteenth, Eleventh, Seventh, Sixth, and Fifth Streets, also at Mission Street and at Commercial and Jackson Streets, as well as a final one at the North Point screen-house, where the sewage will enter the outfall pipes.

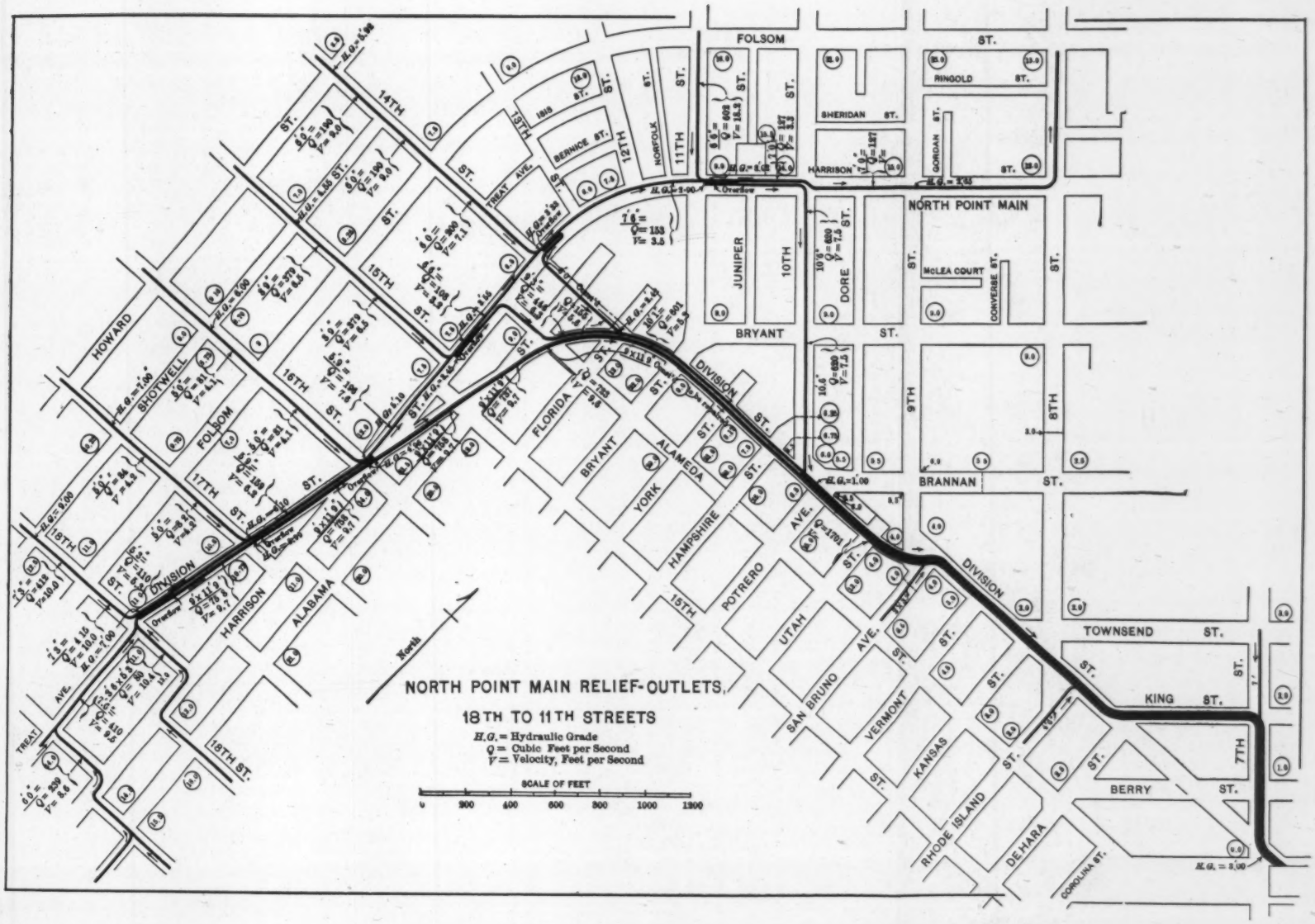
Large conduits will carry the water from each of the storm-water outlets in the most direct line to the open water in Channel Street, or to the bay. Of these conduits, the only one of special interest is the Division Street conduit which will have its head at Eighteenth and Division Streets, but will be the recipient also of the storm-water overflow at Seventeenth, Sixteenth, Fifteenth, and Fourteenth Streets, and at several other points along its course to the head of the open water in Channel Street.

This conduit, shown on Plate XIII, will be called upon to carry ultimately about 760 sec-ft. near Eighteenth Street, and about 1 700 sec-ft. at its outfall end. It will be a conduit 9 ft. high and 11 ft. 9 in. wide from Eighteenth Street to Tenth Street, being paralleled by a second conduit from Alameda Street to Tenth Street. Thence to its outfall, for a distance of about 2 000 ft., the conduit will be constructed with two or three compartments. Three, each 9 ft. wide, will be required ultimately. It may be found advisable, however, to construct only two of these at the outset. Owing to the fact that the floor of the conduit must be kept as high as possible on account of the unfavorable character of the ground along its route, and because the street surface along the line of this conduit is lower than desirable, scant headroom is available, and the conduit has been planned with a flat top of reinforced concrete.

It has been suggested by interested citizens that the main storm-water outfall should be at the foot of Sixteenth Street, and by some it is even contended that the sewage of the entire area that can be made tributary to the intersection of Sixteenth and Division Streets should be carried in a large conduit along Sixteenth Street to the bay.

Taking Fourteenth and Harrison Streets as the initial point for such an outfall, it can readily be seen by a glance at any map of the

PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1127.
GRUNSKY ON
THE SEWER SYSTEM OF SAN FRANCISCO.



city that the distance thence to Sixteenth Street and along Sixteenth Street to the end of Center Street wharf would be twice as great as the distance from the same point along Division Street to the head of the open water in Channel Street. Consequently, a conduit along Sixteenth Street would have less gradient than a storm-water outlet along Division Street. It would have to be of larger cross-section, and the velocities in it would be less than if located along Division Street. In other words, for conduits of similar dimensions, on the two routes, there would be more probability of surface inundation at and near Fourteenth and Harrison Streets if the conduit were located along Sixteenth Street than if located as planned.

The foot of Sixteenth Street does not meet the requirements of a permanent outfall for the sewage of the area that can be made tributary to it. Nothing that can be delivered at North Point should be allowed to reach the water at such a central point on the eastern bay frontage.

Attention may be called to the fact that, in line with this requirement, the sewage of the eastern portion of the Potrero Hills is to be collected in a secondary interceptor which will skirt the northern base of these hills and connect with the North Point main at Ninth and Howard Streets. The storm-water overflow from this secondary East Potrero interceptor will flow in suitable conduits to the Division Street relief outlet.

The North Point main will be in tunnel under Telegraph Hill for 2 500 ft., if it follows Montgomery Street. If located on Sansome Street, the tunnel construction, if required at all, would be much shorter.

In its largest section, the sewer was planned to be circular, 8 ft. in diameter, with a concrete invert and a brick arch. In doubtful ground it is to be supported on piles. Double sheet-piling on each side is to facilitate trenching. The piles used as guide piles for the sheeting are to be left in place as a part of the sewer foundation, and the sheet-piling on either side of the trench is to serve as a part of the form for the concrete invert. Longitudinal steel rods are to be placed in the concrete, and as many piles as may seem necessary for support are to be driven in the trench and capped with the concrete of the sewer invert.

The hydraulic gradient prescribed for the sewer when flowing full will have an elevation of 7.00 ft. above City Base at Eighteenth

and Division Streets, dropping to 3.50 ft. below City Base at the outfall. Its gradient is about 0.0005, or 1 in 2 000. The sewer invert, as originally planned, was to have gradients gradually changing from 1 in 1 800 near Division and Eighteenth Streets to 1 in 2 000 at the outfall.

The hydraulic grade line at the North Point screen-house will be 0.50 ft. above extreme high water of the bay and 3.20 ft. above ordinary high water. The sewer invert at the same point will be about 12.90 ft. below City Base, or a little more than 6 ft. below ordinary high tide.

During the rising tide, therefore, the discharge of sewage will be retarded; during the falling tide it will be accelerated. The effect of this will be a reduced outflow into the flood tides and an increased discharge into the ebb, which is a desirable feature.

The area of the city on the bay front which lies to the eastward of the North Point main will be treated in three subdivisions, from each of which sewage will be pumped into the North Point main.

It is not proposed in this paper to enter upon a minute description of the system of sewers outlined for these and for the many other main and sub-districts, because the problems are in large part the same as are presented in all growing cities. Only those features of the problem will be referred to which appear somewhat unusual or may otherwise be considered of special interest.

Yerba Buena and Mission Flats Sub-District.—The Yerba Buena sub-district, which extends from Telegraph Hill on the north to Rincon Hill on the south, and lies between the North Point main and the bay, is a low, flat, for the most part filled-in, section of the city. It is throughout an important business district.

The storm-waters of this sub-district are to be delivered to the bay for the most part in the sewers now in use, but remodeled as far as practicable to give them continuous slope toward their outfall points.

Sewage proper will be provided for in separate conduits leading to a pumping station between Commercial and Sacramento Streets, a short distance east of Drumm Street. The pumps will send the sewage westward into the North Point main.

The Mission Flats sub-district, 845 acres in extent, which embraces the old Mission Bay region as far as it lies below or east of the North Point main, will also be sewered on the separate system. The

pumping station for this district is to be located north of Channel Street, near Fifth Street. The sewage south of the Channel will be carried under the open waterway in an inverted siphon. The pumps will send the sewage northwestward along Fifth Street into the North Point main.

In both these districts the sewers of the separate system will be ironstone pipes. The smallest lateral sewers are to be 8 in. in diameter, and the minimum gradients on which these are to be laid, with but few exceptions, will be 1 in 200.

The principal objection to the use of the combined system in these districts is the fact that the surface of the districts lies low. Very much of it is at City Base. The efficient drainage of basements has not been possible with the combined system heretofore in use. Better service was demanded. It was necessary to choose between, on the one hand, a combined system of sewers leading to pumping stations, with storm-water relief outlets to the bay and intermittent pumping, and, on the other hand, the complete separation of the sewers from the storm-water conduits. In the former case there would have been times, during rain storms, when the sewers would have been full to a plane well above high tide in the bay, and back-flow into basements unguarded by check valves would have been inevitable. From this situation there was no remedy except either to provide pumping capacity on a combined system great enough to handle the maximum storm-water flow, or to introduce the separate system. The latter plan was adopted, but it, too, has its disadvantages. One is the duplication of conduits in streets already crowded with pipes for gas and water, and with conduits for telephone and telegraph wires and wires for transmitting electricity. It was found necessary in many places to provide sewers for each side of the street.

The main drawback to this system is probably to be found in the fact that some of the streets of the filled-in areas are still in a condition of subsidence. On this point there was very little information available in 1899. It was known that the lower end of Market Street, which had been paved many years before at official grade, was then several feet low, and the various buildings with their sidewalks, which had been constructed at grade, had settled measurably. Nothing was known concerning the continuance of the subsidence. The general opinion seemed to prevail that but little more was to be expected.

The adopted system, under these circumstances, included provision for the support of the sewers on piles. It was proposed to place the sewers in a continuous, pile-supported bed of reinforced concrete wherever the ground was doubtful.

In the light of later information, it now seems uncertain whether any type of construction can be devised that will hold the sewers at grade.

As City Engineer, during 1900 to 1904, the writer made studies of street subsidence. Careful levels were taken at intervals of one year. The results of these studies for the two years, 1901 to 1903, are shown in brief form in Table 18.

TABLE 18.—SUBSIDENCE OF STREETS IN SAN FRANCISCO.

Location.	SUBSIDENCE, IN FEET.	
	1901–1902.	1902–1903.
Davis Street, Market to East Streets	0.05	....
Jackson Street, Montgomery to East Streets	0.03	....
Spear Street, Market to Bryan Streets	0.05	....
Mission Street, First to East Streets	0.05	0.02
Harrison Street, Fourth to Seventh Streets	0.15	0.19
Sixth Street, Howard to Brannan Streets	0.05	0.05
Sixth Street, Brannan to Channel Streets		0.08

From April, 1902, to April, 1903, there was no appreciable subsidence on Davis Street except a small amount at its intersection with Vallejo Street, where the subsidence during the preceding 12 months had been 0.08 ft.

From April, 1902, to April, 1903, there was no appreciable subsidence on Jackson Street except a small amount at Drumm and East Streets, where the subsidence during the preceding year had been 0.06 ft.

From April, 1902, to April, 1903, there was subsidence on Spear Street only at Mission Street, where the subsidence during the preceding 12 months had been 0.06 ft.

The subsidence is not confined to the streets alone, but includes adjacent buildings; even those within the area under consideration that are supported on piles 90 ft. and more in length show measurable subsidence.

The lower end of Market Street in 1903 was from 2 to 3 ft. below

official grade from curb to curb, the street-car tracks supported on piles having settled with the remainder of the street.

As subsidence of the character here referred to includes every structure in or adjacent to the street, no provision for the support of sewers would be adequate to prevent settling. If the sewers are laid to grade there will be sections dipping below grade after a few years, and the dip will increase until after a time the departure from the true gradient may be so great that reconstruction of some of the down-town sections will be necessary.

The earthquake of 1906 has occurred since the subsidence studies were commenced. No information is at hand at this writing as to the effect which the violent shaking has had upon the general elevation of the street surfaces in the made-land districts. Local upheavals of a foot or more, and local depressions of the ground's surface have been noted in various parts of the city. Where these occurred they were more or less destructive to all conduits in the streets, and would have caused serious derangement of the sewer system as planned if it had been in service. It is probable that whatever changes have been produced in the street levels, by the earthquake, have brought them nearer to a permanent condition of stability. At any rate, the probability of the recurrence of earthquakes of sufficient force to prove destructive to sewers is too remote to be taken seriously into account in planning the system. The location of the areas where a derangement is most likely to occur, however, has been pretty clearly indicated by the breaks in conduits of all kinds in the earthquake of 1906, and this knowledge may here and there lead to some modification of the location of a sewer or to some modification of design.

Lower Islais Creek District.—A dual treatment has been planned for the Lower Islais Creek district. From this district sewage proper is to be pumped through a long force main into the North Point sewer. Where there are no sewers now in use the collecting system will be separate from the storm-water drainage; but where sewers have already been constructed to serve for sewage and storm-water combined these are to remain in service. The arrangement at the pumping station, which will be centrally located, will be such that excess storm-water will pass through a relief outlet to Islais Creek. Whenever the excess flow is long continued, which will be coincident with a full North Point sewer, the pumping from the sewers on the combined

system will be stopped. The sewers of the separate system are to reach this pumping station at 15 ft. below City Base. This low grade is made necessary by the large tributary area with low-surface elevation. At a mean stage of water in the receiver at the pump the total lift, including friction in the discharge main, will be about 50 ft.

Only those sewers of this, as of other, districts that lie within improved territory or that are at once required to reach pumping stations or outfall points were recommended for immediate construction. There are some cases in which the sewers recommended for immediate construction lie along streets crossing the marsh and not now graded. In these cases it was assumed that such streets would be ordered graded as a matter of urgency, and that within a few years they would be ready to receive the sewers.

The North Point Outfall.—The screen-house at the outfall of the North Point main is to be a neat structure of brick or concrete. There will be two settling basins of small diameter, to intercept coarse sand and the broken rock reaching the sewers occasionally with the storm-water from the macadamized streets of the outlying regions. These basins are to be arranged so that one or both may be in service. The removal of material from them will be by dredging, possibly by the hydraulic method.

Alongside of the main sewer, just before it enters the screen-house, there is to be a long overfall weir, with crest at the hydraulic grade line, or a little below this, over which any excess flow of storm-water will drop into a low, flat conduit of brick or concrete leading directly to the bay.

The screen-house will be equipped with screens for the interception of rags, cork, and other floating material that should be removed from the sewage before it is discharged into the bay waters. The outfall pipes are to be connected with the settling chambers, and are to be carried at least 400 ft. beyond the head of a solid pier projected for this point. They were planned to be of cast iron, 5 ft. in diameter. It may be found desirable to substitute reinforced concrete. They are to be placed in a trench excavated by dredging to about 50 ft. below low tide.

Hunter's Point Temporary Outfall.

In the case of Hunter's Point main sewer, it is not practicable, owing to the unimproved condition of the streets and their location,

across marsh surface and points of hills, to carry the sewer at once to its final point of outfall. The improvement of the streets along which the sewer is to be built is too far in the future. Their grading for sewers is out of the question on account of cost. The main, therefore, will terminate for a time on the north shore of Hunter's Point, and a discharge will be effected through a submerged pipe well off shore.

Harbor View Sewer District.

The time may come when the demand for keeping the waters of the bay absolutely free from sewage will be so great that, whether offensive to the senses or not, the delivery of sewage on the eastern and northern bay frontage of the city must be stopped. Should this time ever come, the sewers as now planned will remain in service, but pumping will be necessary to send all sewage along the water front to an outfall at Fort Point. The arrangement can there be made to discharge practically all sewage of the bay slope of the city into the ebb tide. In this event the sewage from so remote a region even as Bay View would be delivered by pumping into the North Point main, and would be rehandled by the main pumps located at or near the site of the proposed screen-house. Until this time comes, there will probably always be more or less sewage sent into the bay at points less desirable than those selected as main outfall points.

Until actual experience shows it to be undesirable to send limited amounts of sewage into the bay with delivery into deep water, at a few selected points, the great expense of concentrating all at Fort Point need not be incurred.

It was planned, therefore, to deliver the sewage of the Harbor View District, lying on the northern front of the city to the westward of Black Point, into the bay at the foot of Scott Street. The sewers of this district, as at present, will carry both sewage proper and storm-water. Storm-water relief outlets are planned with a view to bringing existing sewers into use at their full capacity. The drainage system of the district was designed with special reference to the present improvements. The intercepting sewers follow streets along which immediate construction is practicable. When the reclamation of the waterfront property has here been completed, a second line of intercepting sewers of small capacity will be needed. The storm-water relief outlets will serve as local sewers below the upper intercepting line until they

reach the waterfront, where their ordinary flow will follow the lower interceptor and be carried by the same to the main outfall point.

This system of sewerage will have the disadvantage, for the low area close by the waterfront (as yet but partially improved), of affording only imperfect basement and cellar drainage. But the property owners will know the limitations, and can plan their improvements to meet the conditions.

The discharge into the bay will be through a large pipe, about 800 ft. long, terminating in water 40 ft. deep. The capacity of this pipe will be adequate for the ordinary sewage flow, with the usual included small amount of rain-water. All excess flow, as already stated, will go to the bay through storm-water relief outlets.

Richmond Sewer District.

For the Richmond District a sewer project was adopted some years previous to the studies of 1899. The sewage and rain-water are carried in common conduits to an outfall just west of Baker's Beach. At the time this outfall was inspected no trace of sewage at the shore nor of offensive odor could be detected. The strong tidal currents sweeping this part of the shore warrant its use as a point of disposal. The later designs provide for an extension of the outfall farther off shore into deeper water and a completion of such mains as have not yet been constructed.

Other Ocean-Slope Sewer Districts.

The topography of the ocean slope of the city westerly and southerly from Richmond District made a subdivision into four additional districts appear desirable. These are the West Richmond, Upper Sunset, Lower Sunset, and Ocean View Districts. The last named includes the Lake Merced region, and extends southward to the County line.

The problem of handling the sewage of these districts, which for the most part will in time be densely populated, is complicated by the fact that the ocean beach, extending for several miles southward from the Cliff House near Point Lobos, is a pleasure ground of the people, and must be preserved undefiled.

This consideration led the writer and his associate engineers to recommend the separate system of sewerage for all these districts,

and a delivery of all sewage proper into the ocean, off Point Lobos, where it will be so diluted as to disappear completely. From this point, moreover, the oceanward flow is away from and not toward the beach.

Storm-waters were to be collected in separate conduits with outfalls into the ocean at selected points. The principal outfall points tentatively suggested for storm-waters were at the foot of J Street and at the foot of X Street.

It is understood that the plan of treatment here outlined for the Ocean Beach drainages is not to be adhered to. It is now proposed to construct at once an interceptor for both sewage and storm-water which will discharge into the ocean at the Point Lobos outfall point already referred to. Sewage will be liberated in deep water well off shore in the vicinity of Mile Rock.

The rapid improvement that is taking place on the ocean slope has made it appear undesirable to install the works for the temporary delivery of sewage from a West Richmond pumping station into the Richmond mains.

It will be possible to place the proposed main sewer of the ocean-slope districts along V Street to Forty-fifth Avenue, to T Street, to Forty-sixth Avenue, to R Street, to Forty-seventh Avenue, to J Street, to Forty-eighth Avenue, and along Forty-eighth Avenue into the tunnel under Fort Miley Heights.

A low-level sewer will follow the Great Highway northward from X Street to a junction with the main interceptor. Velocities in the low-level sewer are estimated at 2.8 ft. per sec.

All the ocean-slope districts, except portions of Richmond, were but sparsely settled in 1899, but a rate of growth was anticipated that made it imperative to provide means at an early date for a disposal of the sewage from the improved areas. At that time there were improved areas of small extent at and near the beach, both to the north and to the south of the Park, and Upper Sunset was fast becoming popular as a residence district. At Ocean View, too, in the southwestern part of the city, a portion of the built-up area was over the divide on the ocean slope and needed attention.

The ultimate point of outfall taken into consideration for this ocean-slope section of the city was Point Lobos, near Mile Rock. The small service required for a number of years, and the high initial

cost of tunneling under Point Lobos Heights, made it undesirable to attempt to reach this point with an outfall sewer at the outset. For this reason the following treatment of the ocean-slope region was recommended. This has since been modified, as already explained.

Upper Sunset District.—The region lying south of Golden Gate Park, between the Park and the Twin Peaks and Blue Mountain group of hills, as far west as Twenty-ninth Avenue, has been designated Upper Sunset District to distinguish it from that part of Sunset which lies at lower elevation and nearer the ocean.

The sewage of this district, under the plan of 1899, was to be carried northward across Golden Gate Park and delivered into the mains of the Richmond District, with outfall at the foot of Twenty-seventh Avenue, just west of Baker's Beach. The storm-waters were to be carried westward in separate conduits.

At the present time both sewage and storm-water are delivered into the Park, as a temporary arrangement, and are being used for irrigation.

Lower Sunset District.—The main western slope of the city, from Golden Gate Park southward, to near Lake Merced, is known as Lower Sunset District. In 1899 the greater portion of this region was covered by high, irregular sand dunes. Streets had long ago been laid out, but official grades remained to be established. At that time, therefore, it was premature to commit the city to a definite scheme of sewer alignment. Since then, however, this region has been one of rapid development, and no mistake was made in 1899 in recommending the immediate construction of works for sewage disposal for which there is growing need. In this district, as for Upper Sunset District, a complete separation of sewage from storm-water was recommended. This plan has now been modified, as already stated.

West Richmond District.—The region known as West Richmond District is practically a part of the Lower Sunset District, separated from the latter by Golden Gate Park. The sewage of both this and Lower Sunset District was to have been led into the receivers of a pumping station located near the ocean on the north line of the Park. It was to be pumped from there temporarily into the mains of the Richmond District, ultimately through a tunnel under Point Lobos Heights into the ocean near Mile Rock. Under the modified plan, the pumping station is rendered unnecessary.

Ocean View District.—Ocean View District is the area sloping westward from the peninsular ridge, in the southwestern part of the city, toward Lake Merced. It embraces parts of Ocean View, Lake View and Ingleside. It is a sparsely populated district, but needs outfall sewers at once because its drainage is toward Lake Merced, which is still in use as a source of water for the city and will probably be used indefinitely as an emergency source.

The storm-waters of this district will necessarily follow natural drainage lines westward to points where they can be intercepted for diversion past Lake Merced. Some temporary works for such diversion, with permanent tunnel outfall, have already been constructed by the Spring Valley Water Company.

Sewage proper will be collected in separate conduits, and will be carried in a closed main to a discharge into the main sewers of the Lower Sunset District.

Pumping Stations.

The details for the pumping stations were worked out by Mr. H. C. Behr, Mechanical Engineer. Centrifugal pumps were prescribed for all pumping stations, although, for use at the West Richmond station, vertical, crank-driven, triple-plunger pumps were taken into account, to be used until the low delivery through the tunnel at Point Lobos could be effected. Under the modified plans there will be no West Richmond pumping station.

The receiver capacities were fixed, in a measure, to fit the available ground area. If necessary, they may be added to in the future. The receivers were planned circular in outline, in triplicate for each station, and it was assumed that all would have to be carried below the water-table, and that the pneumatic method of construction would be necessary. The receivers at each pumping station are to be interconnected and arranged for use separately or in conjunction. The sewage, before entering the receivers, is to be screened. Ventilation is to be provided by a shaft or chimney, in which a small fan blower, with a multiple injector nozzle, is to create a draft.

All pumps are to be belt-driven. Small units are suggested for the pumps in order to secure the greatest possible adaptability to all requirements. The motors were only tentatively selected for cost-estimate purposes, it being apparent that gas, oil, and electric motors are

all well suited for the work required. Steam engines require more space and a greater cost of attendance. They are objectionable on account of the smoke nuisance, and are not as well suited to the intermittent character of the work that may be required.

The force or discharge main from each pumping station is to be of cast iron. Additional facts relating to the pumping stations are given in Table 19.

TABLE 19.—Sewage Pumping Stations.

Name of station.	Required capacity, in second-feet.	Total lift, including friction, in feet.	Ultimate horse-power required.	Present horse-power.	Capacity of receiver, in gallons.	Length of force main, in feet.	Diameter of force main, in inches.
Yerba Buena.....	6	31	42	35	60 000	2 100	16
Mission Flats.....	15	35	120	80	60 000	4 000	18
Lower Islais Creek.	12	65	180	88	60 000	4 700	18
Bay View.....	5	30	35	14	40 000	1 650	8

Types of Sewers.

Concerning the types of sewers to be used, but little need be said. Ironstone pipes are to come into use up to diameters of 2 ft. Sewers of the next larger size, and up to 3 ft. 6 in. by 5 ft. 3 in., are to be egg-shaped. Larger sewers are to be circular, except as stated in the case of the main relief outlet to Channel Street, which is to be flat—the lower end in two or possibly three compartments, as already explained, each nearly rectangular, 9 ft. wide and 8½ ft. high.

The egg-shaped sewers and the large circular sewers were planned to have concrete inverts with reinforcement to give added strength and stability. They were designed without vitrified brick invert lining.

It is understood that the plans have recently been modified somewhat in the matter of sewer types. The use of reinforced concrete is to be extended. All sewers of greater diameter than 2 ft. are to be constructed of this material. Vitrified brick is to be used for invert lining of some of the sewers.

It is to be stated in this connection that recent examinations by the City Engineer of concrete sewers in Sixth Street and H Street that were constructed about five years ago show that these sewers, except for a slight wear in the invert of the H Street sewer, are in excellent condition. No deleterious effect of sewage upon the concrete could be noted.

Whether the tendency, so generally apparent throughout the country, to build the sewers with concrete shells of minimum thickness, which is sure to be attended with many failures, is to be followed or not, remains to be seen.

The Bond Issue.

The report of 1899 was accompanied by a cost estimate covering the construction of the sewers and storm-water conduits immediately required. The expenditure of $4 600 000 was recommended.

At an election, held soon after the adoption of the report, bonds in this amount were voted to carry forward the work. Before these bonds were issued there was a modification of the form of government in San Francisco. Theretofore the city was subject to control by the State Legislature. Legislative enactment, in the early history of San Francisco, had consolidated City and County. Amendments to the Consolidation Act were thereafter the convenient means of securing privileges in the city of laying out new streets, and of prescribing and limiting the powers of the Board of Supervisors and the other City and County officials.

The charter which was adopted in November, 1899, modified all this, and, among other things, created a Board of Public Works, with usual duties; but this charter, which went into effect on January 8th, 1900, made no provision for carrying to completion the proceedings relating to bond issues commenced under the Consolidation Act. The matter was carried into the courts, whose decision was to the effect that the sewer bonds already voted could not be issued. New proceedings were necessary.

During the first few years under the new charter such matters as the bonding of the city for improvements moved slowly. Each step had to be subjected to the test of the courts. It was not until 1903, therefore, that a new order was passed by the Supervisors directing the City Engineer again to prepare plans for a comprehensive system of sewers.

In accordance with this requirement, the plans of 1899, with but slight modification, were re-submitted by the writer, then City Engineer, under date of June 30th, 1903. The cost estimate was increased because it covered sewers not recommended for immediate construction 4 years before, and because there had been considerable advance in the price of materials, and particularly in wages. The estimated cost of

the sewers and accessory structures, then recommended for construction, was $7 250 000.

The question relating to the issuance of sewer bonds in this amount, together with bonds for other purposes, in the aggregate about $17 000 000, was submitted to the voters in November, 1903. The bonds for sewers were again voted. But the bonds as authorized were to bear only 3½% interest. This low rate of interest coupled with the fact that the government of the city had meanwhile passed into the hands of officials who did not command public confidence, made the sale of these bonds impossible. At length further attempts to sell bonds of this issue were abandoned, and the City Engineer was called upon for a new report. This was after the great calamity of 1906. Restriction of the sewer work to the main lines of sewers was demanded, in order to keep the bond issue as low as possible. The latest recommendation, therefore, does not cover all the sub-mains that were included in the recommendations of 1903, nor even the full system of 1899.

The final submission, in May, 1908, to the voters, of the proposition to expend $4 000 000 on the sewer system again resulted favorably by a vote of about 15 to 1. Under the bond issue thus authorized the systematic construction of the projected main sewers and interceptors has now been commenced.

DISCUSSION.

Mr. Hammatt.

W. C. HAMMATT, ASSOC. M. AM. SOC. C. E. (by letter).—In the writer's opinion, there are few cases when it is advisable to handle storm-water and sewage together; and it is particularly objectionable when the discharge is below water. For a given district, variations of the quantity of sewage proper are slight, while the storm-water will vary from nothing for a great part of the year to a quantity many times in excess of the sewage on some occasions.

In a main emptying below water level, therefore, and designed to carry both storm-water and sewage, the velocity would be so slight when sewage alone was being carried, that sedimentation would take place. Cases are conceivable where the sedimentation would be so great during prolonged dry seasons that the mass would be too great for the first storm to remove, and serious damage might be done to the outfall piping and the lower sewer districts.

It might be claimed that the relief outlets would take care of all the storm-water if the main pipe were obstructed in this way, but if such a thing should happen, and the storm were of a low intensity, the sewage which would take the relief outlet would be diluted so slightly as to be offensive, and as the relief sewers are to empty in exposed places, this would be objectionable.

The writer believes that sewage proper and storm-water should be handled in separate conduits and under separate systems. The sewage should be discharged below low-water level in such a manner as to produce proper dilution and removal by currents, as described by Mr. Grunsky. The storm-water should be discharged above tide level, when possible, at the most convenient beach or pier head. The outfall for sewage proper should be of such cross-section that the velocity against the back-water would never fall below 0.5 ft. per sec.

Mr. Dieck.

ROBERT G. DIECK, ASSOC. M. AM. SOC. C. E. (by letter).—In connection with the design of an improved sewerage system for San Francisco, it may be of some interest to state the method of design followed in the storm-water drainage system of Manila, P. I. To understand clearly the essential geographical features of the city, a few remarks are necessary. The city is situated at the mouth of the Pasig River, on the east shore of Manila Bay, on an alluvial plain formed of sand and gravel, mingled with loam, brought down from the range of mountains which forms the main drainage slopes of the Island of Luzon. In addition to the major stream, the river has two main branches which take off within the limits of the city, and serve as additional reliefs in times of high tide and floods in the river. One branch extends north and the other south, and both are of about equal importance. There are several smaller streams, which intersect these

Mr. Dieck. branches and the main stream, forming a network of canals of more or less importance, which makes the system of waterways practically interdependent. There are also several minor connections to the bay, but these are of lesser importance. Except at extreme low water, practically all the streams are navigable for small river boats, and furnish a ready means of transportation. The larger streams have been dredged where shoaling has occurred.

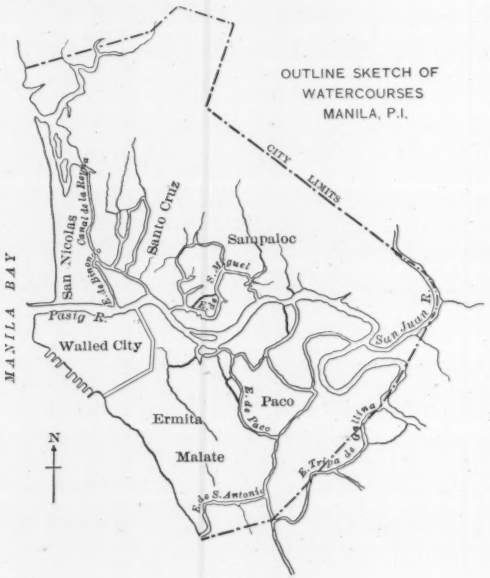


FIG. 12.

The ground surface rises from about 2 m. above mean low water in the southern and western portions of the city to about 5 m. above the same datum in the northeastern portion, where small foot-hills are encountered. Practically 75% of the area of the city lies within these elevations. As the average range of the tide does not exceed about 1 m., water is encountered in portions of the town at a depth of 1 m. below the street surface at high tide. Because of this low elevation, the water-table is almost constantly near the surface in large areas of the town; and, for the most part, in the business and residence sections, trenches more than 1.5 m. deep require to be pumped. As these sections required attention at once, it was soon seen that the shallow trenches made necessary by the high water-table would influence seri-

ously the design of the sewers. During the rainy season, which occurs from about June to late in October or even to early December, as a consequence of the torrential rains, the water-table rises, and, in some parts, almost coincides with the street surface, making dry trenching an impossibility without pumping. Mr. Dieck.

The river has a depth of about 14 ft. at the east end of the harbor (obtained by dredging), decreasing to about 6 ft. at extreme low water at the eastern limits of the city. The main branches have a maximum depth of about 8 ft. Gaugings of these streams and the river have not been made, but, in times of flood, when the current has a velocity of nearly 10 ft. per sec., the discharge of the river must be at least from 50 000 to 60 000 sec-ft. For the most part, the *esteros* (as the branches are called) are sluggish and rather shallow, exposing large portions of their beds at low water. Their frequency, however, makes them admirable recipients of storm-water, and makes further subdivision into drainage districts unnecessary.

Before it was possible to arrive at a satisfactory solution, it was evident that the entire question of street grades would have to be considered, and that some definite plan for these, in opposition to the former, haphazard plan, would have to be adopted. To this end, careful levels were taken throughout the city, and the data were marked on the city plans. Preliminary computations clearly showed that to avoid excessive fill, which, under local conditions, would have proven extremely costly, and to protect property interests against serious damage following violent changes in grade, a flat gutter grade was absolutely essential. After long discussion, this grade was fixed at the absolute minimum of 0.002. It had been observed that water would stand, on the most carefully constructed work laid to a grade less than this, and as all stagnant water is objectionable in the tropics, it was held that this feature should govern the work, even if in street construction the item of filling were large. As a further limiting factor, it was fixed arbitrarily that the street surface at no point should be less than 0.75 m. above the highest tide. The limits set varied for the various districts. Although it was known that all the low districts would be subject to overflow during high floods in the river, it was still held that some reasonable height of street surface should be fixed in all cases, in order to improve, wherever possible, a naturally bad condition.

Another limitation was in reference to cover. After some observation of 24-in. concrete pipes, under heaviest city traffic, the minimum cover was fixed at 0.60 m. Up to the present this cover has been sufficient. For reinforced sewers no limit was set except an amount sufficient to clear the paving. Under the worst conditions, therefore, with the further provision that the sewers should not be laid below mean low water, because of the likelihood of silting, the sewers had to be

Mr. Dieck. designed to fall entirely within from 1.15 to 1.25 m. It was essential, also, that the space occupied by the sewers should be kept at the lowest limit in order to provide for other structures in the narrow streets, these, in some cases, being only 8 m. between building lines, with a clear roadway of only 6 m. At every turn, therefore, certain definite restrictions were to be observed, and also the strictest economy in space and head-room.

A study of the rainfall records was then made. Fortunately, the admirable records of the Philippine Weather Bureau were available, and were examined for maximum intensities for a period of at least 16 years. The records clearly showed that intensities of more than 3.00 in. per hour were not uncommon. Table 20 is a list of typical storms which were considered as probably the worst to be encountered.

TABLE 20.—TYPICAL STORMS IN THE PHILIPPINE ISLANDS.

Date.	Total rainfall, in inches.	Duration, in minutes.	Excessive rainfall, in inches.	Hourly intensity, in inches per hour.
Apr. 29, 1905	2.56	14	0.79	3.50
May 21, 1892	2.36	30	2.36	4.72
June 8, 1905	2.26	10	0.98	5.90
" 15, 1891	2.16	8	0.59	4.43
Sept. 18, 1887	2.06	30	1.57	3.05
Aug. 26, 1903	2.05	30	1.62	3.24
July 19, 1899	2.02	15	Record imperfect.	
Sept. 15, 1891	1.98	12	0.785	3.94
July 16, 1890	1.98		Record imperfect.	
Nov. 16, 1891	1.96		Record imperfect.	
June 1, 1902	1.91		Record imperfect.	
Aug. 6, 1889	1.89	7	0.59	5.51
June 30, 1889	1.86	20	1.18	3.54
Sept. 14, 1898	1.86	17	0.985	3.48
Aug. 27, 1888	1.85		Record imperfect.	
Sept. 17, 1888	1.85	35	1.79	3.07
July 13, 1904	1.81	10	0.73	4.37
Aug. 28, 1897	1.81	16	1.57	5.90
Mar. 9, 1894	1.80		Record imperfect.	
July 12, 1904	1.75	10	0.79	4.72
Sept. 6, 1896	1.73	15	0.985	3.15
July 23, 1888	1.68		Record imperfect.	
Aug. 26, 1886	1.65	15	0.59	2.36
May 29, 1891	1.65	10	0.787	4.72
Sept. 20, 1904	1.62	10	0.71	4.25

Storms having an intensity of 2 in. were common throughout the rainy season, and occurred when the ground was thoroughly saturated, as well as during the drier times. It was evident, therefore, that no relief could be expected from ground storage, but, as the runs in the sewers were short, it was decided that the maximum flow could not occur until about 10 min. after a severe rain had commenced. Table 20 shows that ordinarily the very heavy rainfalls did not last more than from 20 to 25 min., so that if the maximum discharge occurs as stated, and provision is made for lesser rainfalls, there can be flooding only for 10 or 15 min. As there are no cellars or basements in the

city, provision against back-water in the sewers was not required, and it was considered that with safety and at the expense of only a temporary inconvenience, flooding of the roadways would be permissible during unusual storms. Under such cases the sewers would run under a head, which would increase their capacity. Mr. Dieck.

It was then decided to design the sewers under the assumption of a 2-in. rainfall, discarding the excessive storms from consideration, relying on the sewers under pressure to discharge the excess, and permitting the flooding of the roadways for 15 min. In the design no account was to be taken of the increased capacity of the sewers due to this head, the excess being viewed merely as a factor of safety in the design. This compromise was made only after the closest preliminary trials and after it was clear that the restricted head-room, flat grades, and the necessary economies in expenditures required some concessions to be made. It was also kept in mind that during severe storms the river and the *esteros* were subject to overflow and that this could not be prevented by any possible provision in the sewers. The results obtained under this assumption, while not satisfactory in all cases, appear to be justified, for no serious flooding had occurred in 1907, two years after the first sewer was built under the general plan.

At first, a run in the gutters of 120 m. had been fixed as the maximum in the built-up sections, but later observations on the gutter flow showed that this was excessive, and the maximum was reduced to 100 m. Sewers with inlets at this distance apart give better satisfaction, and flooding has been lessened. All inlets have direct connections to the sewers, and are provided with silt basins and grating covers.

In the designs, Kutter's formula for discharge, with sewers running full, was used, the values of n being taken as follows:

0.011 for cement plaster finish,
0.013 for clean brick and glazed pipe,
0.015 for local stone.

A velocity of 3 ft. per sec. was sought wherever possible, so as to lessen the likelihood of silting, which had always been the greatest difficulty in the old sewers built under Spanish direction. All drainage areas were kept completely separated, and the connection between the old sewers built by the Spaniards was stopped in order to make the direction of flow definite. To a great extent, the deposits were undoubtedly due to this error in design, and the immediate betterment in the condition of the old sewers following the removal of the connections was made evident by the increased discharge. Wherever possible the old Spanish sewers were utilized, the sides and inverts being made smoother by cement plaster.

In the absence of any data on the character of the run-off, an assumption of 75% was at first made; but this was found to be ex-

Mr. Dieck. cessive in large sections and deficient in the closely built-up parts of the Walled City and in the business sections. The final run-off coefficients were fixed as follows:

Heavily built-up	0.90
Ordinary business and residential	0.75
Suburban	0.50

Undoubtedly, these high coefficients lessened the flooding of the streets and are ample for all future work. It is difficult to compare the assumptions with the results obtained by Emil Kuichling, M. Am. Soc. C. E., or with those by Mr. Grunsky, but it is clear that there has been no serious error in the designs.

The *esteros* are in need of improvement, but their betterment would involve the city in such enormous expenditure that no definite plans to this end have yet been evolved. A large amount of dredging is necessary, and the streams must be confined to definite channels by walls. A preliminary section for a light shallow wall of reinforced concrete was designed, but the preliminary estimates of cost were so appalling that this could not be recommended. These *esteros* are the natural reliefs of the river, and their improvement is of the highest importance in the scheme of storm-water drainage. At times the velocity of the current does not exceed 1.0 ft. per sec. and no scour of the bed is possible. A deepening of the streams would result in increased flow, and their present unsanitary condition would be removed. It is quite probable that there must be some control of their currents, by gates or otherwise, if the reduction of deposits is to be secured, but, at present, there seems to be no necessity to consider that in the design of the confining walls.

From these remarks it will be clear that the drainage problem of Manila has many nice points to recommend it to the attention of hydraulic engineers, and while no claim is made to any originality, it is believed that the solution is somewhat out of the ordinary.

Mr. Gregory. CHARLES E. GREGORY, ASSOC. M. AM. SOC. C. E.—This elaborate paper presents an unusually able theoretical solution of the problem of run-off in storm-water sewers, and is most valuable as an academic discussion of the subject. In the speaker's judgment, however, the author has not given enough weight to the influence of storage on the surface of roofs and streets. After making certain assumptions for a hypothetical case, this part of the problem is very ingeniously and correctly solved for that case. The assumption is made that the rain reaches the sewer from the roofs and streets in maximum volume within 5 min., which is similar to the assumption often made by supposedly careful investigators and writers. The author, however, shows a much better appreciation of what actually occurs than they in that he takes account of the storage on the surface for the time

assumed. For many roofs and a few street surfaces this assumption of 5 min. is no doubt approximately true, but a careful consideration of what should occur, theoretically, and a few observations of what actually does occur, will indicate a wide variation from the assumed 5 min., and demonstrate that storage on the surface, particularly for short, intense storms, has a much greater influence on the run-off than is indicated in the paper. Mr. Gregory.

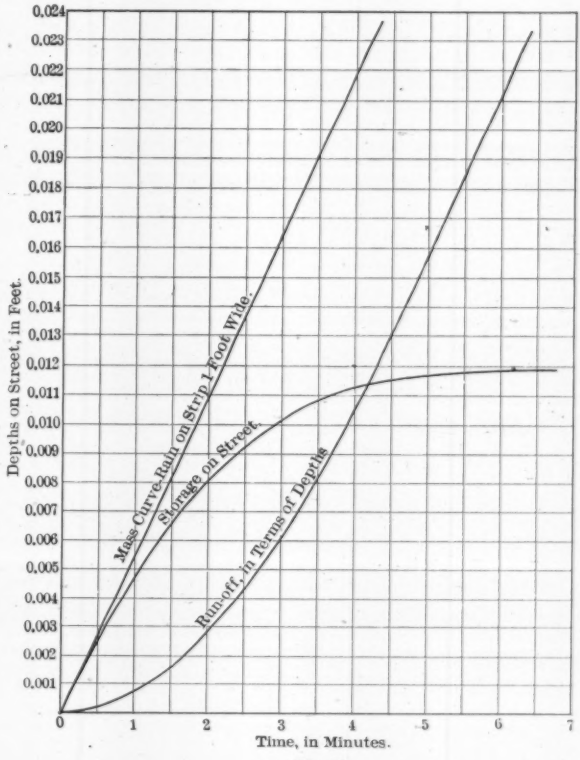


FIG. 13.

It is not the speaker's purpose to attempt to solve this very interesting and intricate problem completely, but simply to present a few studies of storage on catchment areas, which have been made for the solution of special problems, and to show their relation to the general subject.

Mr. Gregory. Referring to a paper by the speaker and the discussion thereon by A. Marston, M. Am. Soc. C. E.,* it was there brought out that:

"*First.*—The water falling on the so-called 'impervious' area of an ordinary sewer water-shed does not all run off as fast as it falls, but part of it accumulates in increasing quantity on the surface during downpours of moderate length, such as cause the maximum discharges from sewer districts of ordinary size.

"*Second.*—As the storm continues at the same rate, the ratio of run-off from the 'impervious' area to the rate of rainfall increases, owing to the increased depth and velocity of the surface flow toward the sewer, until finally, if the storm lasts long enough, the rate of run-off from the 'impervious' areas becomes equal to 100% of the rate of rainfall."

Rough observations indicate that the depth of water which accumulates during heavy downpours on an ordinary street, sidewalk, or roof, is greater than is generally assumed, and that the mean velocity of flow in the gutters is not as great as it is usually computed.

The accumulation of rain on a street has been considered theoretically in the following manner:

The laws and coefficients of flow are not very well determined for very shallow depths, and the impact of the falling rain and other factors materially retard this flow. It was assumed, however, that Kutter's formula was applicable, and, to allow for these retardations, a high value of n was assumed. As an asphalt street, from which the greatest flow may be expected, has a low crown, it was assumed, as has been observed to be roughly true, that the accumulation of depth would be about uniform over its entire surface. From these assumptions, the diagrams on Fig. 13 were deduced. Three curves are there shown. The mass curve of depths for the entire rainfall is for a rate of 4 in. per hour. Points for the other two curves were obtained from the expression, $t = \frac{S\ (H_2 - H_1)}{I - \frac{Q_2 + Q_1}{2}}$, by plotting values of H_2 and Q_2 in terms of depths.

t = increment of time, in seconds, for surface to rise from H_1 to H_2;
S = area of unit length of half the street;
H_2 = depth at end of increment of time, t;
H_1 = depth at beginning of increment of time, t;
I = rate of rain, in cubic feet per second, falling on the area, S;
Q_2 = flow, in cubic feet per second, at edge of gutter, due to depth, H_2;
Q_1 = corresponding flow at beginning of time, t, for depth, H_1.

As an extreme example of storage in streets, the following is presented:

* "Rainfall, and Run-off in Storm-Water Sewers," *Transactions*, Am. Soc. C. E., Vol. LVIII, p. 498.

Mr. Gregory.

The run-off from, and the storage in, the gutter of a 24-ft. street, 1 000 ft. long, with a slope of 0.0025 in 1, is computed in the same general manner as for the street surface. The run-off curve is shown on Fig. 14. For a gutter, or a surface with a uniform slope, the formula is modified to take account of the varying depths, as follows:

$$t = \frac{S\,(H_2 - H_1) - (A_2\,V_2\,T_2 - A_1\,V_1\,T_1)}{I - \frac{(Q_2 + Q_1)}{2}}$$

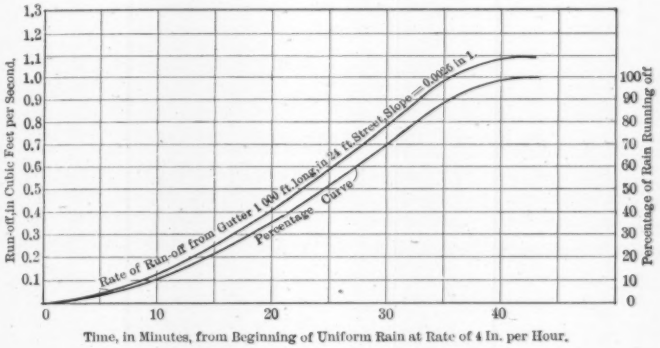


FIG. 14.

In this case, t_1, H_2, H_1, Q_2, and Q_1 have the same meaning as before, but are applied to flow in the gutter.

S = average width of each increment of depth multiplied by 1 000 ft.;

I = 1 000 times the inflow from each unit of length, and is taken from the curve of run-off, Fig. 13;

T_2 = total time elapsed from beginning of uniform rain, in seconds, or until the depth, H_2, is reached;

A_2 = difference in area between the wet cross-section of gutter for the depth, H_2, and the average wet cross-section for the upper reach of the gutter where the depths are variable;

V_2 = average mean velocity in the gutter until the depth, H_2, is reached;

T_1, A_1, and V_1 bear the same relation to H_1 as T_2, A_2, and V_2 bear to H_2.

On Figs. 14, 15, and 16, the rate of the run-off curves is deduced by the same general formula for a rain at the rate of 4 in. per hour, but the storage and inflow curves are omitted, and a curve showing the percentage of rainfall running off is added.

Mr. Gregory.

Fig. 15 is for a street 60 ft. wide and 600 ft. long, with an asphalt pavement and a slope of 0.005 in 1.

Fig. 16 is for a roof 25 ft. by 100 ft., sloping at the rate of 0.02 in 1 toward a gutter across the 25-ft. side.

There are a great many roofs and streets comparable to those assumed for the curves on Figs. 15 and 16. A steeper slope, if paved with stone blocks, would give results equivalent, for flow and storage, to the flatter smooth surface. The Sixth Avenue Sewer District gaugings have been reported by the speaker.* This district contains many 600- and 900-ft. blocks, also many flat roofs, 25 ft. by 100 ft., or larger. It is estimated that it takes 15 min. for water to flow through the sewers from the remote parts of this water-shed to the gauge. In a sewer, the rate of run-off at the gauge point in the sewer from such

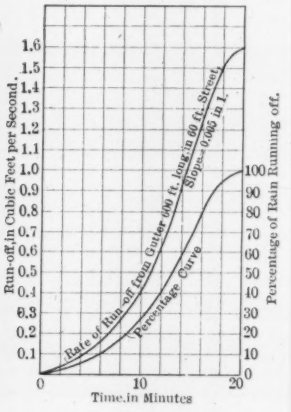


FIG. 15.

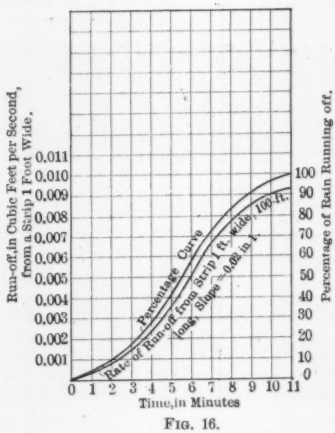


FIG. 16.

roofs and streets, as were assumed for Figs. 15 and 16, may be estimated by assuming that this catchment area is divided into five concentric zones, the outside limits of each being 3 min., 6 min., 9 min., 12 min., and 15 min., respectively, from the gauge. The area of those zones may be taken from the gauge-point outward in the ratio of 1, 2, 3, 4, and 5. The average rate of run-off, expressed as a percentage of the rainfall, for the entire area at the end of 15 min. of uniform rain, at the rate of 4 in. per hour, is found by averaging the percentage of run-off for the average time length of each zone read from the percentage curves on Figs. 15 and 16, respectively.

The averages for the street and roof areas are thus found to be 11% and 48%, respectively. If the catchment area was of such shape

* *Transactions*, Am. Soc. C. E., Vol. LVIII.

that all the zones had equal areas, the percentages would be about 20 and 63, respectively. In a similar manner, a 5-min. time length gives a run-off of about 70% of the rainfall. It is not likely that a rate of 4 in. per hour would be maintained for more than 10 min., and the run-off from the nearby areas would be greatly reduced for the last 5 min. The actual gauged run-off from this area was about 40% for a rain lasting 15 min. This observed percentage may be accounted for by assuming the area to be made up of roofs and streets varying from 5 to 20 min. in time length, taking into account the storage in the catch-basins and sewers, and making allowance for pre-maximum and post-maximum rain rates, the greater influence being exerted by the surface storage. For such rates of rainfall as occur in San Francisco, the velocities of the surface films, due to less depth, would be much less than for a 4-in. rate, and, consequently, the time lengths for similar areas would be greater and have a correspondingly greater influence on the run-off. The speaker wishes particularly at this time to call attention to the great influence exerted by the shape, size, character, and slope of each area, and by the frequency of inlets from the streets. Mr. Gregory.

E. KUICHLING, M. AM. SOC. C. E.—The author deserves the highest credit for having made a more complete analysis of a complicated hydraulic problem than has heretofore been published. The novelty in his procedure is the estimation and consideration of two factors, the first being the quantity of storm-water temporarily in transit on the surface toward the various sewer inlets; while the second is the quantity temporarily stored in the sewers themselves. Both these factors tend to reduce the rate of maximum discharge by making the outflow somewhat less than the inflow until the conduits become entirely filled. Mr. Kuichling.

The first factor is obviously dependent on the extent and slope of the territory tributary to each sewer inlet or catch-basin. If the entire area were impervious and divided into a number of equal or equivalent hopper-shaped parts, each having a catch-basin in its middle point, and in which the time required for the water to flow at uniform velocity from the boundary line to the central basin is t_0 minutes, then the quantity of storm-water which will still be on the surface at the end of such time is shown by the author to be two-thirds of the precipitation on the total area during that time.

It should be noted, however, that uniform velocity of flow over the surface of a hopper-shaped area involves a constantly reducing slope for the successive elementary zones, as the depth of the water is constantly increasing from the periphery to the central catch-basin. Such a condition of surface is rarely encountered in municipalities, and hence it must be assumed that the velocity increases steadily as the water passes successively from the larger exterior elementary zones to

Mr. Kuichling.

the smaller interior ones on the same grade. In general, the velocity in a channel, or on a surface, is proportional to the square root of the depth; and as the depth increases very rapidly on the successively reducing elementary zones of like grade, it follows that much less than two-thirds of the precipitation on a uniformly graded component area will still be on the surface in transit to the catch-basin at the end of t_o minutes.

To illustrate this feature of the case by an example, assume a hopper-shaped area, 1 000 ft. square, having each of the four equal sides graded on a uniform slope to the central catch-basin. Conceive the area divided into five zones, each 100 ft. wide. Beginning at the basin the areas of these zones will be 40 000, 120 000, 200 000, 280 000 and 360 000 sq. ft., respectively, or in the proportion of 1, 3, 5, 7, and 9. Next assume that the rain falls at uniform intensity for a sufficient length of time to allow the run-off from the outermost zone to reach the basin. Now, if it were possible for the velocity to be uniform, as well as the slope, the water flowing over the innermost zone would be about 25 times as deep as on the outermost one. With a uniform grade, however, the velocity will increase from zone to zone, and the depth in the vicinity of the basin will be materially less than the amount just mentioned, since the time of transit is greatly reduced. If the velocity in each zone varies with the square root of the depth of the flow, the time of transit from boundary to center will be less than three times as long as that across the outermost zone, and the quantity of water remaining on the surface at the end of such time will be considerably less than two-thirds of the precipitation during that time.

In urban districts, the catch-basins or sewer inlets are generally much less than 1 000 ft. apart, on the average, and the time required for the run-off from the boundaries of the tributary areas to reach the inlets is usually not more than 10 min., as the water soon reaches the gutters, in which the flow is much more rapid than on a plane surface. During this period of time, the precipitation may be 0.5 in. in depth over the entire territory, and perhaps one-half the water may remain on the surface in transit to the inlets, but, at its termination, the surface storage comes to an end if the rain continues as before, and the run-off will thereafter proceed at its full rate to each inlet, since the storage on the surface is not cumulative when the intensity of the rainfall is uniform. For large districts, in which the time of passage through the longest line of sewers is 30 min. or more, during which time a precipitation of 1.5 in. may occur, the retention of a depth of 0.25 in. on the surface during the first 10 min. of the downpour is not of much significance.

The same may also be said with respect to the storage in the network of sewers, as the cubical contents thereof do not often exceed

an average rate of 1 000 cu. ft. per acre, including house connections. This capacity is equivalent to a depth of 0.28 in. of rainfall; but, as the whole of the precipitation does not reach the sewers, owing to various causes of loss, it may be assumed that the capacity is equivalent to the run-off from a precipitation of about 0.40 in. In heavy rainfalls of comparatively long duration, however, this storage capacity will usually become filled some time before the end of the storm, and surcharge of the sewers will ensue unless their discharging capacity is adequate. Mr. Kuichling.

There can be no doubt as to the propriety of taking these two factors of surface storage and conduit storage into consideration for small districts and short rainfalls, as the aggregate effect may often be very appreciable; but, in the case of large urban districts, having times of transit through the longest line of sewers of more than 30 min., it seems to the speaker that the aforesaid effect will be lost in great degree.

The essential factor in the problem is the relation of the intensity of the precipitation to its duration. Usually, this is estimated more or less arbitrarily from inadequate data, and without an exact knowledge of the extent of territory covered by the rainfall at the recorded intensity. In most cases, also, the extreme observations are disregarded, and the rainfall curve is adapted to the more frequent downpours. If the sewers are proportioned closely to such a curve, there will be no margin of capacity, and it may be questioned whether a moderate reserve capacity, such as would be gained by leaving the above-mentioned two factors out of account, is not a desirable feature. It would certainly be interesting if the author would indicate quantitatively how the full consideration of the said two factors would affect the design of a large sewer system in the Eastern States.

In regard to the rainfall curve that should be used in designing sewers, it has long seemed to the speaker that only those durations should be taken into account for which the intensity or rate per hour was uniform, or nearly so. The object is to ascertain the probable maximum run-off from districts of different magnitude, or duration of flow through the longest line of sewer. Now, it is obvious that the run-off from a homogeneous surface will become largest when the rainfall is greatest and is likewise distributed uniformly over the entire area for a sufficient length of time to allow the water from the outermost boundary to reach the point of observation at the foot of the territory; hence it is of primary importance to deduce from the rainfall data the maximum uniform intensities for different periods of time.

The next step is to determine whether this relation is limited to areas of any particular magnitude, or whether it is possible to obtain a larger rate of run-off at the foot of an extensive area by using

Mr. Kuichling.

precipitations of varying intensity, but having an aggregate duration equal to the time of flow through the longest line of sewer. In general, it will be found that, if the relation between the maximum uniform intensity and its duration has been established correctly, the rate of run-off at the foot of a homogeneous territory of any size will be greatest when the entire area is covered by a rainfall of the highest uniform intensity corresponding to the length of time required for the water to flow to the point of observation from the outermost boundary. Deviations from this rule, however, may possibly occur when the area is not homogeneous, and in such cases one must be governed by the given or assumed conditions of the particular problem.

An instance of non-homogeneous territory is afforded by the following example: Let A equal the number of acres at the foot of a district, with a run-off factor equal to f_1, and the time of flow through the longest line of sewer equal to t_1 minutes; B, f_2 and t_2, C, f_3 and t_3, etc., equal the same elements for the next succeeding component areas; also, let the maximum uniform intensity of the rainfall, in inches per hour (or cubic feet per second per acre), be expressed by $I = \frac{a}{t+b}$, where t is the duration, in minutes. The rates of run-off, in cubic feet per second, from the several component areas, at the end of the time $T = t_1 + t_2 + t_3 + \ldots$ will then be $q_1 = f_1 A I$, $q_2 = f_2 B I$, $q_3 = f_3 C I$, etc., and their sum at the foot of the district will be:

$$Q = (q_1 + q_2 + q_3 + \ldots) = I (f_1 A + f_2 B + f_3 C + \ldots) = \frac{a}{T+b} (f_1 A + f_2 B + f_3 C + \ldots).$$

If the district were entirely homogeneous, we would have $f_1 = f_2 = f_3 = \ldots = f$, and hence, also:

$$Q^1 = I f (A + B + C + \ldots) = \frac{a f}{T+b} (A + B + C + \ldots) = \frac{a f N}{T+b},$$

where N denotes the total area of the district $= (A + B + C + \ldots)$.

It will be noticed that if the run-off factor, f, represents the true mean for the whole district, the values of Q and Q^1 will be equal, thus indicating that it is immaterial whether the surface is homogeneous or not.

To pursue the subject a little further, conceive the territory to be homogeneous and divided into equal squares along the sides of which the run-off from each square, or aggregation of squares, moves with equal velocity, v_o, in feet per second; also let the two consecutive sides of each square have an aggregate length of 60 v_o, corresponding to the path traversed by the run-off in 1 min. At the end of T minutes, the number of squares contributing to the discharge at the lower corner of the first square will be $T \frac{(T+1)}{2}$; and, as the area of each square

Mr. Kuichling.

is $m = (30\ v_0)^2$ square feet, the total tributary area, in acres, will be

$$N = \frac{T(T+1)\,m}{2.43560}, \text{ with } Q = I f N = \frac{a\,f m}{87\,120} \times \frac{T(T+1)}{T+b} = c \times \frac{T(T+1)}{T+b}.$$

We thus have Q as a simple function of T, the form of which shows that Q increases directly with T and attains its greatest value only when T is greatest, or when the entire area contributes to the discharge. This result is due to the peculiar form of the expression for the factor, I. With some other form for I it may happen that Q will become a maximum for a value of T less than that corresponding to the time for flowing through the longest line of sewer in the district.

From this brief analysis of the run-off problem, it appears that attention should be directed mainly to the proper form of expressing the maximum uniform intensity of the rainfall, I, in terms of its duration, t, in minutes. By the use of this factor, the indefinite question of seeking a maximum, Q, with rainfalls of variable intensity during the total time, $T = (t_1 + t_2 + t_3 + \ldots)$, will be entirely avoided, as the former necessarily embraces all combinations of the latter.

The next matter of importance is the investigation of the proper value of the run-off factor, f; but, as the author has not given the results of his own experiments, there is no reason for discussing it here at considerable length. Few direct and reliable measurements of the run-off from a given territory and uniformly distributed rainfall appear to be available. In some cases the rainfall is observed at only a single point in or near the district, and no proof of uniform distribution is offered, while, in other cases, both mean intensity and distribution are estimated. It is very desirable, therefore, to increase the number of accurate measurements for surfaces of different character, and to include therein the actual time of passage through the longest line of sewer, as well as the time required for the water flow over the surface from the outermost boundary to the nearest catch-basin or inlet.

To exhibit the range in the estimated values of the run-off factor, f, or the ratio of the run-off to the rainfall at the time of maximum discharge, the following data are submitted:

For roof surfaces assumed to be water-tight... f =	0.70 to 0.95
" asphalt pavements in good order.....	0.85 to 0.90
" stone, brick, and wooden block pavements with tightly cemented joints.....	0.75 to 0.85
" same with open or uncemented joints....	0.50 to 0.70
" inferior block pavements with uncemented joints	0.40 to 0.50
" macadamized roadways.....	0.25 to 0.60
" gravel roadways and walks.....	0.15 to 0.30

Mr. Kuichling.

For unpaved surfaces, railroad yards and vacant lots	$f =$	0.10 to 0.30
" parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil		0.05 to 0.25
" wooded areas or forest land, depending on surface slope and character of subsoil		0.01 to 0.20

Others do not attempt to make close estimates of the different kinds of surface in an urban district, but content themselves with general average values of f, as follows:

For the most densely built-up portion	$f =$	0.70 to 0.90
" the adjoining well built-up portion		0.50 to 0.70
" the next adjoining residential portions, with detached houses and buildings		0.25 to 0.50
" the suburban portions, with few buildings		0.10 to 0.25

Similar variations are found in the estimates of maximum intensity of rainfall for which provision is to be made in the sewers or other drainage channels, even in neighboring localities where the rainfall has the same character and average annual depth. In view of such large differences in practice in the choice of f and I, there is little encouragement for the adoption of refinements in the general theory of the subject.

Mr. Allen. KENNETH ALLEN, M. AM. SOC. C. E.—It is a good thing to place on record the conditions governing the design, and a description of the essential features, of an important improvement of this kind.

Of the fifteen cities of the United States having in 1906 a population of more than 300 000, all are situated near bodies of water of such size that up to the present their sewage is discharged into them in a crude condition. Baltimore, however, in order to protect the oysters of Chesapeake Bay, is constructing a purification plant, and in order to avoid an undue pollution of the water, Chicago, San Francisco, Boston, and Washington have spent large sums to secure more favorable points of outfall. Without question, in view of increasing populations and a growing sensibility to cleanliness and sanitation, the next two decades will see extensive improvements in most of the other cities in the list.

Of these fifteen cities, four are on the Great Lakes, five (including Chicago) drain to the Mississippi or Ohio Rivers, and six deliver their sewage almost directly to the ocean. San Francisco, however, is especially favored in being able to locate her outlets—ultimately, at least—in strong currents of deep water which trend off shore and soon pass out to sea. New York and Boston are also located on great tidal bays, but the waters of Upper New York Bay oscillate between the Hudson and East Rivers and the Narrows in close proximity to

a dense population for several days before being finally dispersed in the Lower Bay or ocean, while Massachusetts Bay is studded with islands, and the shores are lined with towns and resorts, so that great care has been required in selecting points of outfall to avoid offense. Mr. Allen.

With a discharge into large bodies of water there should be sufficient dilution to prevent putrefactive conditions, and the problem resolves itself into one of rapid dispersion. This can be secured with a single outlet where the current is swift in comparison with the volume of sewage. At San Francisco, as at Boston, interception and a discharge at a few well selected points, after the removal of the grit and coarser floating matters, has been decided on as the best plan. To secure a more rapid dispersion, multiple outlets are provided in the Elbe at Hamburg, and have been proposed for the Passaic Valley Sewer in New York Harbor; but where cities are located on sluggish streams, it is sometimes preferable to maintain numerous independent outlets in order to afford opportunity for a more rapid mixing with the water. The question of interception to one or a few outlets, as at Buffalo or Louisville, or a direct discharge at numerous points, as at New York and Philadelphia, is a local one, and must depend on the circumstances; but it may be said that the tendency is toward the former plan for large cities, so that it has generally come to be looked on as the proper procedure.

The question as to the value of float experiments in locating sewer outfalls is sometimes raised. The speaker believes them to be of great value as indicating the direction in which sewage may be expected to flow under similar conditions. It is true that a surface float furnishes no measure of bottom velocities, which may be quite contrary in direction; that the course taken by one float may be quite different from that of one set adrift a few minutes later, although under conditions which appear to be identical; and that it is influenced more or less by the wind; but in answer it may be said:

First.—A surface float represents the motion of that portion of the current which comprises the greater part of the sewage, through the tendency of the latter to rise and spread out over the surface, diffusion in a vertical direction being comparatively slow; and if bottom or mean velocities are desired these may be determined, approximately, by the use of double or rod floats.

Second.—Although the trace of a single float will not represent a true average result, yet it will show the path that sewage discharged at the starting point might take; and from a sufficient number of records the probability of its going in any given direction may be predicted with considerable certainty.

Third.—A float is influenced by the wind; but this, with properly designed floats, is mainly through surface currents induced by the wind, and it is the trend of the surface currents which determines the

Mr. Allen. path that sewage would follow. The float, therefore, gives the desired information.

In questions involving dilution, depending on the net discharge of tidal and upland waters, the speaker does not consider results from floats as reliable as those obtained by current meters, as it is difficult, by means of the former, to integrate the motion of the filaments properly in a vertical section. Even with the current meter it is a long and expensive operation, if reliable results are to be obtained on tidal streams. An ingenious apparatus for facilitating such work was devised some years ago by Mr. Luigi d'Auria, and used by him on some public work undertaken under the direction of the late William Ludlow, M. Am. Soc. C. E. This device consisted of a pole which could be lowered vertically to the bottom of the stream from the side of a pontoon. An endless cord which passed through pulleys near the top and bottom of the pole was used to carry a 2-in. ball of half the specific gravity of water to the bottom. At a given signal the ball was released, and, rising to the surface at a speed which a computation showed would be uniform, and then being caught by a floating grillage, the mean velocity was obtained by dividing the measured horizontal distance traversed by the time taken to rise to the surface.

Continuous records of single floats covering several tides are valuable in determining the range of tidal oscillation and the resultant movement seaward, but, to secure information regarding the probable dispersion of sewage, the plan used by the speaker, on Chesapeake Bay, of setting a large number of floats adrift at short intervals and locating them as frequently as practicable—each being identified by a number—is an excellent one to adopt, being rapid and economical.

The importance of a careful consideration of local rainfall records is well illustrated in the case of San Francisco, where, if Eastern practice had been blindly followed, the size and cost of the sewers would have been greatly increased. In this respect it would be interesting to learn why the particular form of rainfall formula was adopted, and whether the more usual and simpler form, $I = \frac{a}{t+b}$, would not have answered equally well.

In forecasting the flow of sewage from a given district, the infiltration of ground-water is often an important but uncertain factor. The daily volume has been stated variously in gallons per mile of sewer, per acre or square mile drained, per capita of population, per linear foot of sewer joint, and per square foot of surface. The rational way is to consider infiltration as a function of the sewer surface, if of continuous concrete or brick, and of the linear foot of joint in the case of pipe or other sectional forms of construction.

The perviousness of the soil affects the rate of leakage to sewers. In New Orleans, George G. Earl, M. Am. Soc. C. E., Superintendent of the Sewerage and Water Board, found that:

"In open and unpaved areas we are probably getting double the amount estimated at times of very heavy rains, particularly when they follow a long dry spell, which has cracked open our clay soil, with deep and frequent fissures. Mr. Allen.

"Whether this flow is all seepage or partly due to flooded surfaces where manholes exist and where, even, some one may open a manhole cover occasionally, we cannot tell as yet * * *."

The allowance for infiltration was 0.003 sec-ft. per acre or 31 800 gal. per mile per day, and an estimate made after construction showed this figure—which is the maximum used for San Francisco—to be substantially correct.

In studies made for the sewerage of Baltimore, in 1897, Messrs. Hering and Gray, the Consulting Engineers for that work, recommended an allowance ranging from 0.00024 to 0.0018 sec-ft. per acre.

J. N. Hazlehurst, M. Am. Soc. C. E., secured a somewhat exceptional result* with an 8-in. vitrified pipe laid in 2½-ft. lengths, with a mortar joint of 1 part Louisville cement and 2 parts sand. This was kept covered for 6 hours with a damp cloth, and then immersed for 24 hours. The infiltration was then found to be only 1 329 gal. per mile per day.

X. H. Goodnough, M. Am. Soc. C. E., reported* that on 137 miles of pipe sewers in Massachusetts, varying from 8 to 30 in. in diameter, the infiltration was about 40 000 gal. per mile per day before connections were made, and that it amounted to 70 000 gal. per mile per day on 700 miles of sewer, but that 80 000 gal. per day is estimated in the extensions of the sewers of the Metropolitan District of Boston.

In Malden, Mass., the infiltration was 50 000 gal. per mile per day, 70 gal. per capita per day, or 0.015 sec-ft. per acre.

D. H. Maury, Jr., M. Am. Soc. C. E., mentions experiments made in Peoria, Ill., on a 24-in. vitrified conduit laid in 2-ft. lengths, with 1:1 Portland cement mortar joints, in which the infiltration found by numerous measurements was generally from 8 000 to 35 000 gal. per mile per day, although, in some instances, the figure exceeded 60 000 gal.

The amount of infiltration, of course, will depend largely on the ground-water level, 30 000 to 50 000 gal. per day for each mile of well-laid pipe sewer being probably a fair allowance under ordinary circumstances, but where the exclusion of ground-water is important, this may be reduced by special precautions, and where the sewer lies above the ground-water level there will be a leakage from instead of into the sewer.

Alexander Potter, Assoc. M. Am. Soc. C. E., has secured good results by making joints with sulphur, sand, and pitch, and the speaker has done this by the use of tar pitch and cement, kneaded

* *Engineering News*, August 27th, 1903.

Mr. Allen. by hand to the consistency of dough, as has been the practice for a number of years in the construction of sewers in Atlantic City. Some 20 years ago the speaker attempted to secure a water-tight joint with natural (Black Diamond) cement under furnished specifications, but he was unsuccessful. More recently, however, he has laid about ½ mile of 6-in. and 8-in. pipe, from 0 to 9 ft. below ground-water level, under the usual specifications of Portland cement mortar with a yarn gasket, which, after repairing about a dozen small leaks, the contractor made practically water-tight. To secure impervious work in this way, however, is more difficult and costly than it should be, and the speaker believes that some bituminous joint, poured like a lead joint, may eventually supersede cement—as it already has done to a certain extent abroad—and will furnish a more elastic joint, and, at the same time, a more economical result.

In the case of brick sewers, infiltration varies greatly with the quality of the work and the depth below ground-water level, but there are few definite data to serve as a guide.

Owing to the general substitution of concrete for brick in larger sewers, there is less infiltration than formerly, and this may be done away with practically by careful grading of the aggregate, wet mixtures, and continuous work. In the speaker's experience the greatest difficulty is at the springing line, where the joint between arch and invert, in spite of the bonding key, offers the least resistance to the entrance of external water under pressure. Infiltration in concrete sewers has been prevented entirely by water-proofing laid between the concrete and a brick lining, but the extra cost of the water-proofing and the lining is hardly warranted in order to prevent the comparatively small amount of leakage which would otherwise occur. In a reinforced concrete sewer built at Waterbury, Conn., the infiltration under a head varying from 4 to 12 ft. was found to be 2.08 gal. per day per square foot of surface.

The poor alignment, size, and shape, mentioned by the author as existing in the old San Francisco sewers, have been similarly noted in Boston, New York, and Baltimore, and may be found in most cities large enough to have had sewers more than 50 years old. As a rule, smaller municipalities are freer from such absurdities; but, as an adopted standard design, the old San Francisco 3 by 5-ft. intersection, constructed uniformly 10 ft. below the surface, irrespective of topography or connecting grades, is unique.

The author alludes to changes in the elevation of bench-marks. This, to the extent of 0.10 ft. or more, may be due to upheaval by frost, or by a lowering of the ground-water level on account of the construction of drains, the sinking of wells, etc., depending on the character of the soil, unless the foundation extends to bed-rock. Aside from this, variations have been known to occur, due to more general

movement of the earth's crust. Large areas are known to be involved in these movements, such as the gradual sinking of the New England Coast, which, as John R. Freeman, M. Am. Soc. C. E., has pointed out,* is taking place at the rate of about a foot a century. Mr. Allen.

But there is reason to believe that there may be other alterations in the relative elevations of points within such short distances of each other as to affect the bench-marks of a municipality.

Some time ago, Horace Andrews, M. Am. Soc. C. E., brought a case in point to the attention of the speaker in connection with the "re-levelling of a number of points of the Bavarian precise levels which were originally determined in the interval between 1869 and 1873 and the mean levels taken in 1897."† These were both levels of the highest order, with refinements as to accuracy of standard and permanent marking of bench-marks, which, unfortunately, are often disregarded.

"The discrepancies revealed were about equally plus and minus and in only one or two cases did there seem to have been some gross change in the foundation of the building on which the mark was placed.

"The differences in this Bavarian work were found by touching on nearly fifty points scattered over a large region. * * * Discrepancies of from 13 to 14 mm. seemed to be common, while several were as large as 15 to 17 mm. and two were of 41.5 mm. and one was 86.7 mm. In addition to these was the gross change alluded to of 233.5 mm. in one point."

About 1894, precise levels were run in connection with a general topographical survey of Baltimore then being made by Colonel H. T. Douglas, and numerous bench-marks were established. Twelve years later the speaker had occasion to test a good many of these, and, although precise levels were not run, the discrepancies noted were checked so carefully that he was convinced that numerous changes had occurred during the intervening years. This opinion was confirmed by results reached on another division of the same work, where it was observed that the greatest discrepancies occurred where the slopes were most abrupt. Discrepancies of this kind might appear to be due to a difference in the graduation of the rods, but as these had been compared carefully with a standard, the conclusion reached was that this was not the cause. The apparent errors of elevation noted varied up to several hundredths of a foot.

It would be interesting to learn the general experience of municipal engineers who have had the opportunity of checking benches established by precise methods after the lapse of several years.

WALTER N. FRICKSTAD, ASSOC. M. AM. SOC. C. E. (by letter).— During the past two years, Oakland, on the east shore of San Francisco Mr. Frickstad.

* Report on the Charles River Dam, 1905.

† *Zeitschrift für Vermessungswesen*, August 1st, 1900.

Mr. Frickstad. Bay, has spent about $750 000 on culverts and sewers carrying storm-water, and additional sewers have been, or are about to be, designed. The general method of determining the capacity is adapted from one of Mr. Grunsky's reports on "A Sewer System for San Francisco," substantially outlined in this paper as the "Method of 1899." To those in the city engineer's office this seemed to be a clear, logical, and simple method of making the computations, after having determined certain preliminary assumptions, involving questions of expediency and policy. The claim, therefore, that an improved method has been evolved is of special interest to the writer, who is connected with the city engineer's office in Oakland.

It must soon become apparent to anyone who attempts to design a practical and economical storm sewer that he has attacked a difficult problem, very unsatisfactory from a mathematician's point of view. There are inherent uncertainties which leave the final solution largely a result of the designer's judgment. The principal questions, all or several of which apply to any drainage area, are as follows:

1.—In a growing community, a correctly proportioned storm sewer or culvert will not only serve existing conditions, but conditions as they will exist at some future date. Shall that date be ten, twenty, thirty, or a hundred years hence? A mathematical answer to this question, obtained from interest or sinking fund tables, is not satisfactory, because too many other considerations are involved. This is especially the case when funds must be obtained from a reluctant city governing board, or by the approval of two-thirds of the voters at a "bond election," or over the protests of a large proportion of the property owners to be specially assessed.

2.—What will be the population of the district at the end of the period selected? Estimates can readily be made, but on final analysis, the answer is pure guesswork.

3.—What is the relation between run-off and population, or, to make the question simpler, between impervious area and population? This can be approximated, but the inaccuracy is indicated by the fact that the figures are usually given as 25, 50, 75, or 100 per cent.

4.—Assuming a certain population, or a certain proportion of "impervious area," how much of the area is truly impervious, as the term is used in sewer design? For example, what proportion of the roof area is connected directly to the sewers? To what extent are macadam streets and gravel walks actually impervious?

5.—What will be the effect of pervious area on run-off? This will vary extremely within a single district, according to the character of the top soil, the subsoil, and the degree of saturation at the beginning of a critical period. Authorities differ widely in their statements of percentages; and, as a matter of fact, the percentage will not be the same for any two districts. This, however, for excessive storms over small areas, is one of the minor considerations.

Mr. Frickstad.

6.—What will be the effect of varying grades within the district, both surface grades as they exist, and hydraulic grades as they vary from moment to moment? Very little attempt has been made to answer this question.

7.—What will be the effect of variations of rainfall from moment to moment during the limits of the critical storm?

8.—What shall be assumed as the maximum rainfall rate? Few cities have records showing rainfall rates for short intervals covering the past forty or fifty years, which data are necessary before a satisfactory curve can be plotted. In default thereof, most cities do the best they can with incomplete records, allowing considerable weight to the records of cities of like conditions. Even if the records be complete, is it not fairly certain that at some time all known rainfall rates will be exceeded?

Curves of total yearly rainfall, the proximity of the two cities, and general conditions would seem to indicate that rainfall rates in Oakland should be the same as in San Francisco. On March 30th, 1906, however, P. F. Brown, M. Am. Soc. C. E., observed rates which were higher than any recorded for San Francisco. Table 21 is a comparison of his limiting rain curve, and the San Francisco curve.

TABLE 21.

Duration, in minutes.	Rates, in Inches per Hour.	
	Oakland (1906).	San Francisco (to 1908).*
5	2.60	2.16
10	2.00	1.50
20	1.41 (Measured)	1.02
30	1.10 (Measured)	0.83
40	0.94	0.72
50	0.81	0.65
60	0.74 (Measured)	0.60

* From Mr. Grunsky's paper.

9.—Shall the sewer be designed for the extreme run-off, or shall we deliberately permit flooding of the streets, with resulting damage, once annually, or once in three, five, or ten years? This, which also may modify the answer to the first question, is partly a matter of engineering economics, but is more closely related to the funds available. It is a very practical question, especially under the separate system and in suburban districts, and may modify the capacity of the sewer several hundred per cent.

10.—What will be the cost of duplicating the structure, cutting off part of the district, or otherwise furnishing relief at some future time? The answer to this may modify materially the answer to the first question.

Mr. Frickstad.

11.—What retardation of the water may be expected from the shape of the ground surface? This question is partly involved in the fourth, fifth, and sixth questions, but deserves a separate statement. Average ground, and the improvement thereof, consists of a series of hollows, runs, ridges, and hills. It may be said that the rain, as it falls, gathers in the low places and flows thence over a series of weirs. These weirs vary infinitely in shape and length, and, as the pond changes in depth, the depths over the weir or the velocity of approach, or both, vary extremely. This question is especially important in a small drainage area, and its influence on design can never be determined mathematically.

12.—Where cold weather may be expected, what allowance should be made for a combination of frozen ground, a large body of snow, and a warm rain?

13.—What shall be the final factor of safety?

An examination of these uncertainties, even omitting those which might be settled by a careful census of the population and a physical examination, will show that the capacity of a sewer designed for the ultimate development of a district, with unlimited funds available, to provide absolute protection, might exceed by several hundred per cent. the capacity of a sewer sufficient for existing conditions, providing relief for all times of the year excepting a fraction of one hour.

The proposed new method provides for definite procedure by introducing the factor of storage in permeable soil. Before applying the method, however, it is necessary to make assumptions as to the degree of saturation at the beginning of the storm; and to observe or assume a rate of permeability—at best an average of an infinite number of rates which will actually occur in the given area.

The new method also assumes that one must take into account one of the delays to which water is subject after it falls upon the earth. Omitting for the present, any doubt about the correctness of the theory, it should be noted that these delays arise from many causes, and, even as a whole, are not one of the largest factors in determining capacity.

In two specific cases in Oakland the conduit capacity per acre drained has been examined. In the first case, called the "Grove and Jones Streets storm sewer," at the corner of these two streets, the conduit is a 4½-ft. circular concrete sewer, carrying 87 cu. ft. per sec. The ultimate area to be drained is 175 acres. The present conduit capacity is about 27 000 cu. ft., and the ultimate capacity, on the completion of the upper end, will be about 50 000 cu. ft. Taking the author's assumption that the conduit will be half full at the beginning of the period, the volume to be filled ultimately will be 25 000 cu. ft., being 286 cu. ft. per acre, or a layer of 0.079 in. over the area. The probable rainfall rate, based on a 30-min. period, is 1.10 in. per hour, of which 0.55 in. would fall in the period. The increase of water in the conduits, therefore, would be about 14% of that which falls in the

district. This is a fair illustration of conditions in a residence district with moderate grade. Mr. Frickstad.

The second case has somewhat less population, and has steep grades. At the corner of Eleventh Avenue and East Nineteenth Street the conduit is a 24-in. pipe, carrying 32 cu. ft. per sec. The area drained is 67 acres. The present conduit capacity above the point mentioned is about 3 300 cu. ft., but ultimately, extensions at the upper end may double this. The area available for filling during a storm, therefore, may be taken as 3 300 cu. ft., being about 50 cu. ft. per acre. This would be a layer of 0.0138 in. over the area. The probable rainfall in 20 min. was taken as 0.47 in., of which the available area in the conduits, when extended, would take about 3 per cent.

The engineer, by the exercise of judgment, or through force of circumstances over which he has no control, must fix assumptions in answer to all the uncertainties enumerated. In view of this fact, undue refinements in any one of them, or in calculations based thereon, seem to be out of place.

Admitting that conditions in a given locality might warrant refined calculations (as, for example, in a fully developed district), there are several points in the theory of the author's new method which seem fallacious to the writer.

Of these points the principal one is the introduction of "conduit storage" as affecting the maximum rate of flow past a given point. This would certainly imply that portions of the water suffer a retardation of velocity in some part of their course. This is not conceivable in a well-designed system where each segment has a free outlet, excepting that it may be retarded by flatter grades on the lower portions. It is not believed, however, that the author considers this retardation in his deductions, as this is taken care of in his discarded "Method of 1899." As a matter of fact, there would be a slight acceleration of velocity during the critical period over that given by the sewer grade, due to a slight increase in the hydraulic grade during the rise of water in the sewer, according to recorded velocities observed before and after the crest of a flood in open streams.

The true action of the run-off may best be explained by reverting to the author's illustration of a small area contributory to an inlet. After stating the assumptions of the illustration, he says:

"The area tributary to each inlet may now be regarded as subdivided into 5 concentric zones, from the innermost of which water will reach the inlet in the average time of $\frac{1}{2}$ min., from the next in $1\frac{1}{2}$ min., and so on to the last one in $4\frac{1}{2}$ min."

* * * * * * *

"The water on the ground at the end of the 5-min. period, therefore, is $\frac{66\,x}{100\,x}$, or two-thirds of the total amount of rain which fell in that period.

Mr. Frickstad.

"Generalizing, it may be said that what is true of an area with circular outline is true of other compact shapes, and that, therefore, two-thirds of the water which falls during any time period equal to the time required by water to reach the inlets from the outer portion of areas drained by each, will, at the end of that period, be still on the surface in transit to the inlet."

A further examination of this illustration shows that on the inner area, at the end of the period, there will be $\frac{98x}{5}$, being practically three-tenths of the whole $66x$. In the second area there will be $18x$, being about 27% of the whole. In the third there will be 23%, in the fourth, 15%, and in the outer, 5 per cent. The average depths of water over the several areas will be as 98, 30, 15, $7\frac{1}{7}$, and 2.

All this increase of water appears in spite of the fact that it has opportunity to flow away freely. It is a misnomer, therefore, to call it "storage," for there is no retardation of velocity under the conditions of the illustration.

Furthermore, the above figures, while showing an accession of water on the areas near the inlet, are not factors in determining the maximum rate of flow into the inlet at the end of the period, which may be truly called the "critical instant." To be sure, the rate can be calculated therefrom, but is, quite independent of all the above, rainfall intensity times area. Or, in actual practice, intensity times area, in proper units, less storage (water delayed in transit) due to causes not appearing in the theoretical illustration.

It may further be said that this illustration is in no way modified if the water be conceived to be wholly or partly concentrated in rivulets or gutters, instead of being spread uniformly over the ground.

If this illustration be a correct conception of a small drainage area, it may properly be extended to cover a larger district, for example, a district which requires 60 min. for the water to flow from the outer zone to the inlet. The quantity of water on the surface at the "critical instant," for which the capacity of the inlet must be proportioned, may readily be determined; but, independently, the flow into the inlet will be intensity of rainfall times area, less the quantity retarded by soaking into the ground, and less the quantity retarded in transit due to the pond-and-weir character of the average ground surface. Here, again, it would seem fair to conclude that the theoretical rate would not be modified if the water approaches the inlet in a uniform sheet, or in rivulets, or in artificial conduits. All such modifications, as well as those of varying grades, are cared for by the assumption that 60 min., no more and no less, are required to make all parts of the area tributary.

At this point it should be emphasized that a storm-water system is merely a convenient underground concentration of water which otherwise would flow over the ground. For the purposes of this discussion, it is assumed to have a course as free as the open ground. Its capacity

Mr. Frickstad.

at each point, therefore, must be at least as large as is indicated by the location of that point in the system of concentric rings described in the illustration. Hence the above statement, that conditions will in no way be changed if the water be conceived to be concentrated in channels, that is, the quantity of water within the ring, and the rate of flow across the area, will be unchanged.

Referring again to the Oakland examples cited, it will be recalled that sewers have been built through the lower ends of the districts. Later, as convenience or necessity requires, the sewers will be extended farther into the upper ends. It is somewhat uncertain how the new method would be applied to these cases. The actual volumetric capacity of the conduits, as designed in each case, is about half the ultimate capacity. Following the new method blindly, it would seem that, the reserve volume being less, the sewer should now be larger than when finally completed. On the other hand, the water is at present flowing over the ground, or on the streets, in courses having an infinite capacity. Consistent with the theory of the new method, this condition should exert a large influence on the rate of flow of the sewer at any point considered. In fact, since part of the system has an indefinitely large capacity, might it not be said that the flow in the constructed portion will be very small because of "storage" in the open portion? As a matter of fact, this will be partly true, but it will be because of actual retardation on the ground and in the gutters.

Furthermore, it would appear, from the theory of this method, that the larger the conduit capacity above a certain point the smaller may be the sewer at that point. Obviously, there must be some limit to this, or it might be concluded that an inlet may be very small because the surrounding gutters and earth have an infinite capacity; and the outlet of a system may be small because the upper end is grossly too large.

The fundamental error, if there be such in the author's deductions, seems to be in the statement preceding Equation 1. That is, while the statement is strictly correct, it is not clear how the total quantity of rain falling on a given area, or the total quantity flowing therefrom in a given period, governs the computations for capacity. It is the maximum flow, that at the "critical instant," at the end of the "critical period," which presumably is required. Of course, if an examination of the average or total run-off is undertaken, for the purpose of determining the maximum, and that course is proved to be the simplest and most accurate, it should be adopted; but the relation of the average to the maximum, in Equation 5, depends on an assumed hydrograph, which is probably not correct for any actual storm.

In brief, the writer's whole doubt of the theory of the new method may be summed up in the question: Does not the "critical instant" come at the end of the "critical period" for a storm-sewer system, all

Mr. Frickstad. portions of which have a free outlet; and can the condition of the conduit at any time previous thereto—or after—have any influence on the capacity at that instant?

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—The conduit storage feature of the writer's solution of the storm-water flow problem will perhaps be made somewhat clearer by the use of a diagram which illustrates graphically what takes place in a drainage basin during a downpour having a duration of t_D minutes.

Let the time period, t_D (which must not be mistaken for the critical time, t_m), be divided into ten sub-periods, each $0.1\ t_D$. It will then be found that for values of t_D, as ordinarily occurring in municipal problems, the total maximum rain, q, may be expected to fall in the sub-periods as follows:

In one sub-period of $0.1\ t_D$ minute's duration	$0.30\ q$.
" " " " " " "	$0.14\ q$.
" " " " " " "	$0.10\ q$.
" " " " " " "	$0.085\ q$.
" " " " " " "	$0.075\ q$.
" " " " " " "	$0.070\ q$.
" " " " " " "	$0.065\ q$.
" " " " " " "	$0.060\ q$.
" " " " " " "	$0.055\ q$.
" " " " " " "	$0.050\ q$.

These values are based on the San Francisco, and on the parabolic, types of rain curves. In order that their departure from a formula of the general type, $I = \frac{\alpha}{t + \beta}$, may be apparent, there are shown in Table 22, in parallel columns, sub-values as determined for a 60-min. period, by the San Francisco formula, by the parabolic formula, and by the Kuichling formula, the latter, $q = \frac{2\ t}{t + 20}$, being based on $I = \frac{120}{t + 20}$.

The sub-periods are arranged in Table 22 as they would appear in a rain of decreasing intensity. They may follow each other in any order. In other words, the rain curve, as established by a recording gauge, will lie somewhere between two curves, the one being the limiting rain curve, representing a rain of decreasing intensity, and the other, the same curve inverted, representing a rain of increasing intensity.

The drainage basin or district under consideration can be subdivided into ten sub-areas such that the time, including the inlet time, which it will take for the water to flow from the limits of the innermost sub-area will be $0.1\ t_D$, from the limits of the next $0.2\ t_D$, from the limits of the next $0.3\ t_D$, and so on. Let these sub-areas be desig-

Mr. Grunsky.

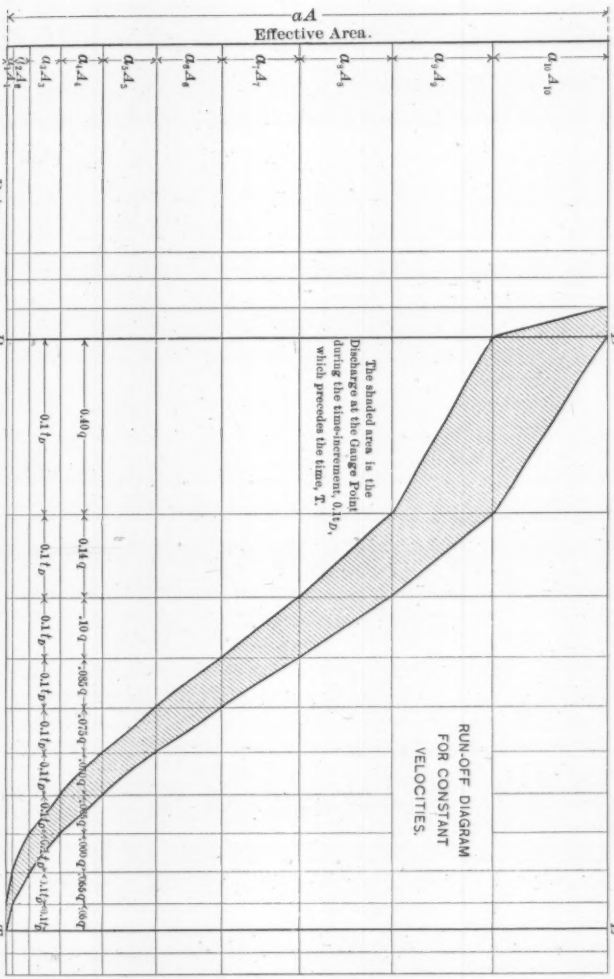


FIG. 17.

Mr. Grunsky. nated, in their order from the innermost to the outermost, A_1, A_2, A_3, ... A_{10}. A diagram, as shown in Fig. 17, can now be prepared. Here, the sub-areas, each modified by its proper reduction factor from Table 6, are platted up from the base line, and horizontal lines are drawn in such a way that the distances between the successive lines correspond to the effective area values, a_n A_n.

TABLE 22.—THE DISTRIBUTION OF THE MAXIMUM RAINFALL IN 60 MINUTES TO SUB-PERIODS OF 6 MINUTES EACH.

The sub-periods are arranged in the order of rain intensities.

Number of the sub-period each 0.1 t_D min.	RAINFALL IN 0.1 t_D MINUTES, WHEN t_D = 60 MINUTES.		
	San Francisco formula.	Parabolic formula.	Kuichling formula, $q = 1.50$.
1	0.29 q	0.32 q	0.31 q
2	0.13 q	0.13 q	0.19 q
3	0.10 q	0.10 q	0.13 q
4	0.080 q	0.085 q	0.095 q
5	0.075 q	0.075 q	0.073 q
6	0.072 q	0.067 q	0.056 q
7	0.067 q	0.063 q	0.047 q
8	0.065 q	0.058 q	0.039 q
9	0.062 q	0.053 q	0.032 q
10	0.057 q	0.050 q	0.027 q

The greatest rain which may fall in any sub-period (0.1 t_D minutes) is now scaled off along some convenient portion of the base line, and vertical lines are drawn from each end thereof. The rainfall in each of the other sub-periods is also laid off along the base line in such manner that the sum of the rain in the ten sub-periods is equal to the total rain in the time, t_D. In the diagram, the sub-periods are shown in the same order as in the table. They may be arranged in any other order, as already stated.

Assuming—though merely for the purpose of illustration—that, throughout the time t_D, when gutters and conduits are carrying considerable water, the velocities remain constant in all parts of the system (being those assumed in calculating A_1, A_2, A_3, etc.), then diagonal lines can be drawn through each rectangle in the diagram, each line representing the progress of the water through a sub-district. The flow of water through the whole district can now be traced by following the successive connected diagonals from the outer sub-district to and through the innermost.

The area, D B T, Fig. 17, is the run-off from the district in the t_D minutes, B to T, which has passed the outfall point at the time, T. The area, D E T, is the water in temporary storage in conduits and on the surface, that is to say, the water which is still in transit.

Mr. Grunsky.

The area between each two successive oblique lines, as drawn in the diagram, from the upper horizontal to the base line, represents the total run-off in the successive increments of time, each of which is 0.1 t_D minutes in duration.

If the assumption of constant velocities were correct, diagrams of the character shown would lead directly to a determination of discharge values and to discharge curves, Fig. 18, but this is not a safe assumption. It has here been made merely as a basis for the preparation of diagrams, such as the one shown, to aid in determining the probable time after the occurrence of the maximum rain intensity when the discharge will be a maximum.

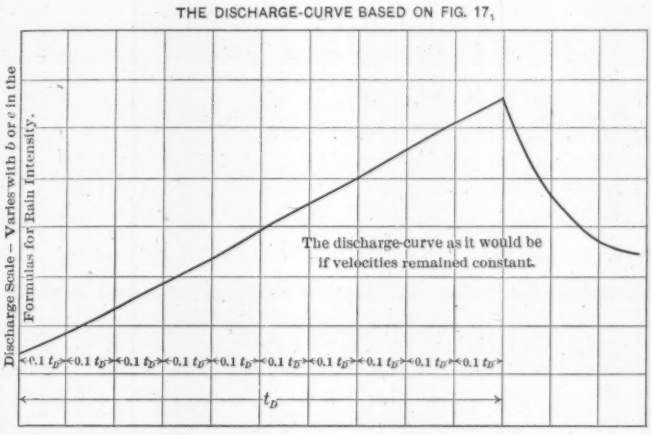


FIG. 18.

The diagram, Fig. 19, serves to illustrate what takes place under the assumption of rain of uniform intensity in the many small areas tributary to inlets. In the case of these areas, the successive sub-areas will increase in extent from the innermost to the outermost about as stated in the paper. This diagram for the progressively increasing sub-areas tributary to inlets, illustrates and confirms the conclusions already presented.

By constructing diagrams, similar to those shown in Figs. 17 and 18, for areas of various shapes, and for rains of various types, a number of discharge curves were obtained, each of which served to indicate, approximately at least, the time at which discharge would be at maximum value. It was found that, for the same value of t_D, the shortest interval between maximum rain intensity and maximum discharge would obtain in the district in which the effective center

Mr. Grunsky. (computed with due regard to the location and effective size of each sub-area) was nearest to the gauge point.

Thus, for a fan-shaped district, having sub-areas which increase from the innermost to the outermost in the ratios (for effective areas) of 1 to 3 to 5, etc., the greatest discharge will take place about t_D minutes after the beginning of the downpour. In districts with uniform effective sub-areas, this time would be about 0.90 t_D minutes, and in districts that are of broad-base triangular shape, the time would be about 0.40 t_D minutes.

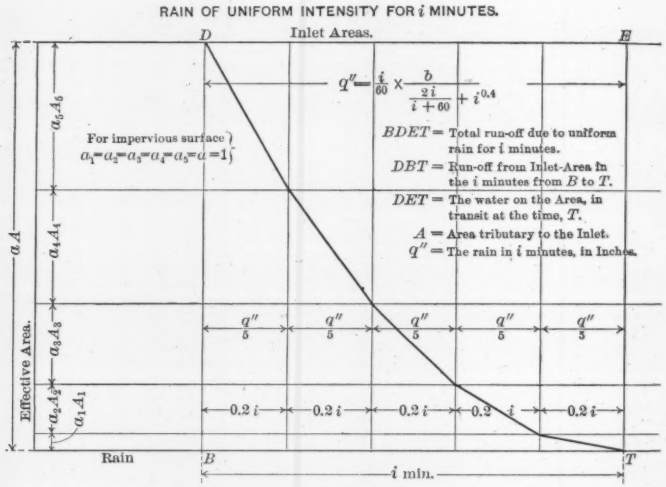


FIG. 19.

This fact suggests that the ratio of t_m to t_D is a function of the shape of the district. The diagram, Fig. 17, is based on a fan-shaped district, the sub-areas increasing with distance from the gauge point.

Taking the foregoing into account, and allowing for the fact that the actual discharge curves will depart somewhat in shape from those determined on the assumption of constant velocities, a formula has been framed in which weight is given to the distance (in time of flow) of the sub-areas from the gauge point. This formula, having proper regard for the permeability of each sub-area, may be written:

$$t_m = \frac{1.40\ t_D}{a\ A}\ (0.1\ a_1\ A_1 + 0.2\ a_2\ A_2 + 0.3\ a_3\ A_3 + 0.4\ a_4\ A_4 + 0.5\ a_5\ A_5 + 0.6\ a_6\ A_6 + 0.7\ a_7\ A_7 + 0.8\ a_8\ A_8 + 0.9\ a_9\ A_9 + a_{10}\ A_{10}) \quad \ldots\ldots\ldots\ldots (63)$$

The formula, if written for five instead of for ten sub-areas, becomes: Mr. Grunsky.

$$t_m = \frac{1.25\, t_D}{a\, A}(0.2\, a_1 A_1 + 0.4\, a_2 A_2 + 0.6\, a_3 A_3 + 0.8\, a_4 A_4 + a_5 A_5) \ldots (64)$$

which may be written

$$t_m = \frac{0.25\, t_D}{a\, A}(a_1 A_1 + 2\, a_2 A_2 + 3\, a_3 A_3 + 4\, a_4 A_4 + 5\, a_5 A_5) \ldots\ldots\ldots (65)$$

For a district which has impervious surface throughout:

$$t_m = \frac{0.25\, t_D}{A}(A_1 + 2\, A_2 + 3\, A_3 + 4\, A_4 + 5\, A_5) \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

And, in special cases (all impervious areas):

1.—For A_1 to A_2 to A_3, etc., as 1 to 3 to 5, etc.,

$$t_m = 0.95\ t_D.$$

2.—For $A_1 = A_2 = A_3$, etc.,

$$t_m = 0.75\ t_D.$$

3.—For A_1 to A_2 to A_3, etc., as 9 to 7 to 5 to 3 to 1,

$$t_m = 0.55\ t_D.$$

In ordinary cases, the subdivision of a district into three sub-areas will suffice. In that event, the formula may be written:

$$t_m = 0.40\, \frac{t_D}{a\, A}(a_1 A_1 + 2\, a_2 A_2 + 3\, a_3 A_3) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (67)$$

It has been shown in the paper that t_m varies with the square of S (Equation 33); consequently, if the foregoing equation for t_m is correct, the storage space, *i. e.*, the conduit contents, should be greater per acre for a district that broadens with distance from the gauge point than for one with its greatest extent near the gauge point. As this is, in fact, the case, it may be regarded as confirming in a measure the conclusion which was reached by another line of reasoning.

The formula for t_m (Equation 67), does not contain S, and therefore it becomes possible to estimate the duration of the critical rain and also the maximum discharge without first approximating the value of S. This is as it should be, because the increase of water in transit during a critical period, whether the water flows on the surface or in underground conduits, varies with the conditions that modify its velocity of flow. These same conditions fix the value of t_D, from which, as shown, t_m can be approximated.

What shall be assumed as the maximum rainfall rate is very properly asked by Mr. Frickstad. The available rain records are usually single-station records. When the rainfalls at several points in the same small drainage basin are combined, the combination should be made, not by averaging results, but by combining the rain curves according to actual time. It is evident that, if there were a number of station records in a district, the composite curve (long-time records being assumed), would show a smaller maximum rain rate than single-

Mr. Grunsky. station records, because the station maxima will not always fall together. This is particularly true for short-time periods. Consequently, when areas are of considerable extent, the probability that station maxima will exceed the maxima that should be used in constructing curves should be recognized, and a proper limiting curve may be drawn that will not fall beyond all the points of observation. On the other hand, a storm may travel down stream in the drainage basin in such a way that the time of maximum rain intensity is later at the gauging point than in the heart of the district. Under such conditions, there may be unusual concentration of run-off waters at the outfall point. This fact, which is of importance in large districts, should be allowed for by giving the rain curve a safe position for the long periods of time. It is well, therefore, to let few if any observed points for periods of 1 hour or more fall outside of the rain curve, while some records, for 5 and 10 min., and an occasional record for other periods less than an hour, above the line may not be objectionable. Conditions, however, may be such that, owing to the high initial cost of a conduit system, it will be unwise to attempt to prevent all inundation. In such case, the rain curve may fall far short of being strictly a limiting curve.

The statement will bear repetition that a reliable rain curve can be constructed for any locality if the maximum rain in some period of time, about one hour in duration, has been correctly determined by observation.

The rain records cited by Mr. Dieck for Manila indicate that a safe limiting rain curve will be obtained by making $b = 17$ or $e = 0.35$. The resulting value of R is about 2.75 in.

Mr. Allen asks why it is necessary to depart from the type of formula for rain intensity which has heretofore found general acceptance. This is necessary because the older type does not, with sufficient accuracy, indicate rain rates or rain amounts for the longer time periods that must occasionally be given consideration in designing storm-water conduits. No formula which is based upon the assumption of a fixed uniform quantity of rain in varying times, or which approximates such a type, will answer. Those who wish to cut loose from all formulas of rain intensity may assume a specific rain distribution to sub-periods of time, as has already been pointed out.

Mr. Gregory is undoubtedly correct in what he says relating to the great variation in the time that it takes water to reach inlets. When this time is assumed to be short—5 min.—the time of concentration is cut down, and the rain rate entering into the formulas is increased. The assumption of a short inlet time was made by the writer in order that the resulting formula may to this extent be a safe formula; but, it is shown in the paper that some variation in the time of flow to inlets will make but little difference in the resulting formula, except

only for very small areas. It is the increase or decrease of the water in transit which counts, and not the amount thereof. When there is no increase or decrease, the run-off is 100%, as quoted by Mr. Gregory, and this is the essence of the writer's reasoning (see Table 5). Mr. Grunsky.

The estimate of time, made by Mr. Gregory, at which the run-off from any impervious surface becomes equal to the rain rate (the rain rate being assumed uniform) is interesting and instructive, but of no apparent application to the theory of run-off presented in the paper. This time is not the time, t_D, which it will take water, during extreme rain conditions, to flow to the inlets. The point in connection with the whole problem which is particularly to be emphasized is that it matters very little whether the storage be within the conduits or on the surface, provided only that t_D is estimated correctly, being the distance to the inlets divided by the velocity of approach to the inlets (no matter whether this be on roofs or across lawns or in gutters), plus the time of flow in the conduits from the outlying inlets to the gauge point.

Data are not as complete as desirable for a proper application of the new formula to the Sixth Avenue sewer in New York City, the flow of which, on August 4th, 1888, has been cited by Mr. Gregory. He states, in reference to this sewer, that it takes 15 min. for water to flow through the sewers from the remote parts of the district to the gauge. He also says that there are many 600 and 900-ft. blocks in the district, and many flat roofs. Assuming 10 min. as the allowance for the time required by water to reach the inlets, then $t_D = 15 + 10 = 25$ min., and, taking $t_m = 0.80\ t_D$, makes $t_m = 20$ min.

The downpour on August 4th, 1888, produced a maximum run-off of 1.16 sec-ft. per acre.

The measured rain is reported to have been at the rate of 3.54 in. for 10 min. For 20 min. it was at the rate of 2.59 in. per hour.*

Using the rate for 20 min. and the formula as developed for a San Francisco type of limiting curve, it will be found that:

$$b = 9.89\dagger$$
$$R = 1.61$$

The district is 90% impervious; therefore, from Table 6, for $t_D = 25$ min.:

$$a = 0.98$$
$$\text{and } d_m = \frac{4.36\ a\ R}{3.82} = 1.71 \text{ sec-ft. per acre.}$$

This is 50% more than the recorded flow, estimated from the water stage in the sewer. The sewer was at the time surcharged. It is doubtful, therefore, whether the value, 1.16 sec-ft. per acre, is the

* *Transactions*, Am. Soc. C. E., Vol. LVIII. p. 464, Plate LXIX.

† If the 10-min. rate of 3.54 in. had been used, the value of b would have been 9.91.

Mr. Grunsky. real maximum that would have occurred if the sewer had been larger. It is possible, too, that the value of t_D used in the formula has been underestimated. It has been assumed that the measured rain rates in this case are reliable.

The parabolic type of rain curve for the same elements above used would make:

$$e = 0.193$$
$$R = 1.50$$
$$\text{and } d_m = \frac{5\,a\,R}{\sqrt{t_m}} = 1.65 \text{ sec-ft. per acre.}$$

No basis can be found for the opinion expressed by Mr. Kuichling, that the effect of storage upon discharge, which he admits for small districts, is lost in large districts. The diagram, Fig. 17, should afford a complete answer to this view. The diagram is applicable without regard to the size of the drainage basin.

The new formula may be regarded as a confirmation of the good judgment heretofore used by engineers, particularly by Mr. Kuichling, in adopting reduction factors to be applied to rain rates when run-off is to be estimated. As long as channel storage, in its effect upon the rate of discharge, was overlooked, it was natural to assume that absorption of water was the main cause for a discharge value less than the rain rate; but, the reduction is, in part, sometimes altogether, as has been shown, due to the changing quantity of water in transit to the gauge point. It remains but to add that the new formula as written, or the formula with some modification of its numerical factor based on better knowledge of the shapes of the discharge curves from areas of all possible outlines, will leave small room for error in the case of impervious or nearly impervious areas. It is not necessary to assume any loss of water from such areas; all is accounted for.

A satisfactory concrete example for Eastern rainfall conditions, asked for by Mr. Kuichling, cannot be given without a study of rainfall data, for which time has not been available.

Of the series of questions asked by Mr. Frickstad, nearly all relating to the run-off under known conditions of rain have been answered in the paper. The conduit capacity which has entered into the demonstration is the storage capacity of an adequate system of conduits. Where conduits are inadequate, or are laid only in portions of a district, the value of S includes all increase of water on the surface as well as that in the conduits. Care should be taken that the storage effect be combined with the surface run-off, and not with the rain. The rain rate and the surface run-off rate are only then equal when the surface is impervious throughout.

Oakland has a somewhat heavier rainfall than San Francisco. In applying the formulas based on the San Francisco type of rain curve,

the value of b should probably be placed somewhere between 4.00 and 4.50. It is quite possible that the parabola will more nearly approximate the Oakland rain curve. Mr. Grunsky.

The term, storage, as used in the paper, is fully explained. The quantity of water in transit is continually changing. It increases during a downpour. The case is exactly the same as though the water were flowing through a reservoir. If this view be taken of the problem, it will be clear that the rain preceding a critical period may have some effect on the flow during the critical period, and, therefore, on the maximum flow. Let it be supposed that there be sufficient conduit capacity above the gauge point to give the conduit the character of a reservoir. It will then be plain that a maximum rain will produce smaller run-off rates if the reservoir be empty at the beginning of the critical period than if it be full. In other words, the most unfavorable critical period will be one preceded by heavy rain; but, on this point, enough has been said.

In order to make clear the effect upon the formula of the assumption that the discharge curve has a shape during a critical period, substantially as shown in Fig. 7, it may be noted that, if the discharge curve had been taken as a straight line from B to T, that is, if it had been assumed that the discharge increases at a uniform rate, then the resultant formula for d_m would have contained a numerical factor, 0.83, instead of 0.71. The probability is thought to lie with the lower value, for all ordinary problems. Special assumptions and resulting modifications of the formula may become necessary when the range in the sub-area sizes is out of the ordinary.

The sinking or settling of certain areas in San Francisco alluded to in the paper, is a local phenomenon, due entirely to compression of the material on which parts of the city have been built. The writer, while City Engineer, commenced the work of precise leveling which is being extended to all parts of the city. In connection with this work, tests were made to find whether certain low streets, the surface of which had once been at official grade, were still settling, or whether they had come to rest. It was found that some of them were still sinking. Neither frost nor change of ground-water level has anything to do with this phenomenon.

Recapitulation.

Having determined a value of t_D from the known extent and topographic features of a district, and having estimated values of three or of five sub-areas, as explained, the following formulas may be used:

$$a\,A = a_1\,A_1 + a_2\,A_2 + a_3\,A_3 \ldots\ldots\ldots\ldots \text{acres.}$$

$$\text{or } a\,A = a_1\,A_1 + a_2\,A_2 + a_3\,A_3 + a_4\,A_4 + a_5\,A_5 \ldots\ldots\ldots\ldots \text{“}$$

$$t_m = 0.40\,\frac{t_D}{a\,A}\,(a_1\,A_1 + 2\,a_2\,A_2 + 3\,a_3\,A_3) \ldots\ldots\ldots\ldots \text{minutes.}$$

Mr. Grunsky. or $t_m = 0.25 \frac{t_D}{a A} (a_1 A_1 + 2 a_2 A_2 + 3 a_3 A_3 + 4 a_4 A_4 + 5 a_5 A_5)$ minutes.

For the San Francisco type of limiting rain curve:

$$I = \frac{b}{\frac{2 t_m}{t_m + 60} + t^{0.4}} \text{ .....inches per hour.}$$

$r = a I$..... " " "

$R = \frac{b}{6.14}$.....inches in one hour.

$S_m = 27.5 a b \sqrt{t_m}$.....cubic feet per acre.

$S_m = 169 a R \sqrt{t_m}$..... " " " "

$t_m = \left(\frac{S_m}{27.5 a b}\right)^2$ ....minutes.

$t_m = \left(\frac{S_m}{169 a R}\right)^2$ ..... "

$d_m = \frac{0.71 a b}{\frac{2 t_m}{t_m + 60} + t_m^{0.4}}$.....cubic feet per second per acre.

$d_m = \frac{4.36 a R}{\frac{2 t_m}{t_m + 60} + t_m^{0.4}}$..... " " " " " "

For the parabolic type of limiting rain curve:

$$I = \frac{60 e}{\sqrt{t_m}} \text{ .....inches per hour.}$$

$r = a I$..... " " "

$R = 7.75 e$.....inches in one hour.

$S_m = 1\,565 a e \sqrt{t_m}$ .....cubic feet per acre.

$S_m = 202 a R \sqrt{t_m}$..... " " " "

$t_m = \left(\frac{S_m}{1\,565 a e}\right)^2$ ....minutes.

$t_m = \left(\frac{S_m}{202 a R}\right)^2$ ..... "

$d_m = \frac{38.8 a e}{\sqrt{t_m}}$..... cubic feet per second per acre.

$d_m = \frac{5 a R}{\sqrt{t_m}}$..... " " " " " "

Values of a, for both types of rain curve, dependent on the value of b, or e, and of t_D, are noted tentatively in Table 6. The velocities which prevail when conduits are full should be used in estimating the value of t_D.

In applying the new formula, all roof areas, street pavements, paved yards, macadamized roads, and the like, are to be considered impervious. The loss of water during short-time periods from such surfaces is small when compared with the quantity of rain. Lawns, open ground, dirt roads, and the like, are to be classed as pervious.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1128

WALNUT LANE BRIDGE, PHILADELPHIA.*

By George S. Webster and Henry H. Quimby, Members, Am. Soc. C. E.

The bridge on the line of Walnut Lane, Philadelphia, over Wissahickon Creek, is notable for the great size of its main arch, for the novelty of some of its features, the character of its material, and the method of its construction. It is hoped that a description of it, with a discussion of the principles of its design, the reasons for its special features, and the lessons learned in the course of its construction will be of interest to the Profession.

The ravine of Wissahickon Creek, in Fairmount Park, is narrow and deep. It separates two populous sections of the City—Germantown and Roxborough—which are both on high ground, and heretofore most of the travel between them has been required to make a long detour, all the bridges being at a low level and the approaches to them too heavy in grade for business hauling.

By Ordinance of July 13th, 1905, City Councils authorized the construction of a high-level bridge on the line of Walnut Lane, the point where the ravine is narrowest and most advantageous for the crossing.

The required length of structure, of course, depended on the elevation of the deck, and as considerable grading of approaches was necessary—cutting for one and filling for the other—the rate of grade to be obtained on the approaches, together with the estimated money value of each percent. of reduced street gradient, entered into the consideration of the problem of dimensions and design. The question was some-

* Presented at the meeting of September 15th, 1909.

what further involved by the fact that on the Germantown side the approach is from three streets. Fig. 1 exhibits the original profiles and the elevations and grades adopted. A portion of the filling for the west approach was obtained from the cut of the east approach, and was carted across the bridge.

The bridge deck was given an ascending grade to the east—1½% to the middle of the main span and 1% from there to the east abutment. This change of grade was made in order to produce a slight crowning and relieve the elevation, or side view of the bridge, of the optical illusion of a dished appearance in the deck over the arch. This crowning is equivalent to a local grade of 1 in 400 on each side.

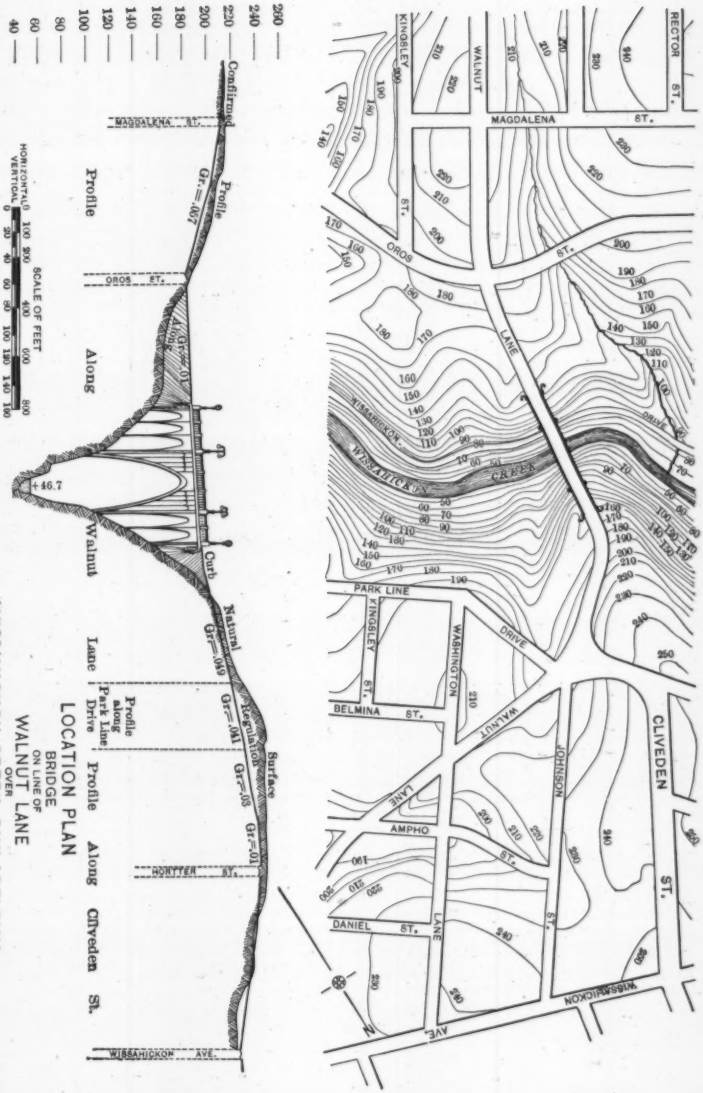
The grades adopted give the floor at the middle an elevation of 147 ft. above ordinary water level in the creek. As the depth of the structure at the crown is 14 ft., the soffit is 133 ft. above the creek and about 103 ft. above the drive, giving an imposing and majestic sweep of arch that is very impressive.

As the two springing points are at the same elevation, the grade over the bridge, in making the spandrel piers on one side higher and therefore thicker, puts extra dead load on that side, and this is counterbalanced by making the walls of the spandrel piers on the other side 4½ in. thicker.

In considering the matter of type of structure and length of spans, it was regarded as very undesirable, for æsthetic reasons, that there should be any pier or column interposed between the creek and the pleasure drive which winds along the west bank at an elevation of about 30 ft. above the water. The situation clearly indicated an arch as the only suitable form of support for the roadway, and the necessity of clearing both the drive and the creek, while making reasonable provision for the future widening of the drive, taken in consideration with the profile of the rocky bluffs, fixed the location of the abutment piers and the span of the main arch, which was made 233 ft. between the vertical faces of the piers.

The extraordinary length of the main span gave to the structure a monumental character, and it became important that the remainder of the design and the material of which the bridge was to be constructed should be in accord with it.

The lowest first cost would have been found in a steel viaduct with a wooden floor, but the character of the surroundings—natural park



LOCATION PLAN
BRIDGE
ON LINE OF
WALNUT LANE
OVER
WISSAHICKON CREEK, PHILADELPHIA

FIG. 1.

scenery of rocky and wooded slopes—would cause such a structure to appear inappropriate and inadequate, and the possibility of future neglect of proper maintenance enhanced the importance of selecting a construction of the most permanent character.

The development of the sphere of usefulness of concrete, and the city's extended experience in the use of it for arches, walls, and floors, with new and improved methods of manipulation and construction, whereby more nearly uniform character and value are obtained, and the adequacy and permanence of the material are assured, together with the great economy obtained through its use, influenced the adoption of concrete as the structural material of the bridge throughout, and it was designed so as to require steel reinforcement at only a few points.

The most economical length of the approach spans was determined by trial to be 60 ft. from center to center of the piers, or 53 ft. clear. The conditions called for three of these spans on the west slope and two on the east slope of the ravine.

The profile of the rocky banks naturally fixed the location of the springing points of the main arch, and gave a graceful proportion between the clear height above the creek and the length of the span.

The type of structure selected is that of the stone arch bridge in Luxemburg, Europe, built in 1904, which is substantially two bridges side by side with the space between them floored over. The type is economical for such a structure, because large masonry arches of a width sufficient for their own independent stability have more than ample strength for any highway live load that may have to be carried, and any increase in width or strength would be utilized only in sustaining the weight of the added material. Thus, in the main span of this bridge, the arch ribs themselves constitute one-half of the whole load to be carried—both dead and live—and, as the live load is less than one-twelfth of the dead load, a single barrel of the full width of the floor would clearly be wasteful because its thickness could not safely be made much less than that of the narrow ribs. The width of the floor is 60 ft., while that of each of the two ribs is 18 ft.—giving together 36 ft. of supporting arch, or only 60% of the floor width. The space between the ribs is 16 ft., leaving 4 ft. of floor to overhang on each side.

Each of the two halves or ribs of the bridge carries two vertical

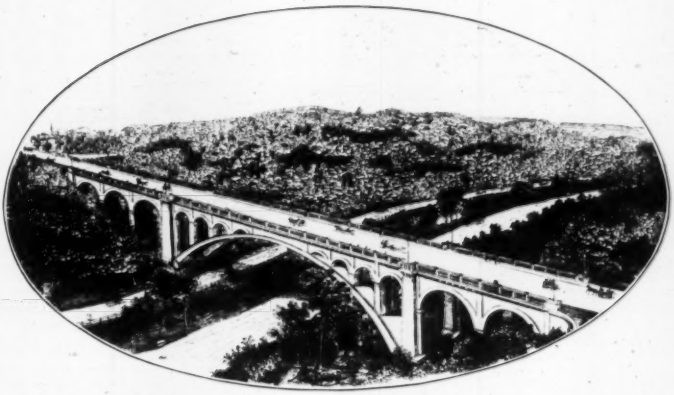


FIG. 1.—WALNUT LANE BRIDGE OVER WISSAHICKON CREEK, PHILADELPHIA.

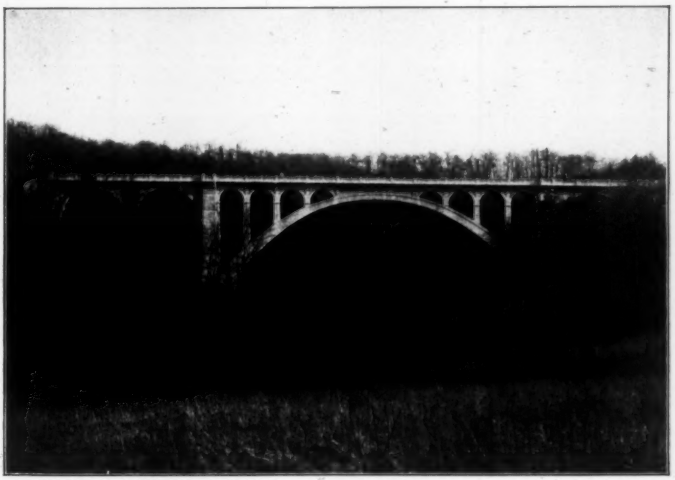


FIG. 2.—NORTH SIDE, WALNUT LANE BRIDGE.

walls, 2 ft. 6 in. thick, which extend up to and carry the floor. These walls are set in 9 in. from the edges of the ribs, giving an actual overhang of 4 ft. 9 in. to the sidewalk floors on the outside of the bridge.

The three aisles thus formed, and spanned by the floor, are 20 ft. and 14 ft. for the middle and side aisles, respectively, from center to center of the walls.

These longitudinal walls are full spandrels over the 53-ft. arches, but on the main arch they are pierced by four 20-ft. arches over each haunch, leaving 60 ft. blank over the crown. These 20-ft. openings are flanged 6 in., in order to give the barrel or rib of each arch a width of 3 ft. 6 in., and pilasters of equal relief, 2 ft. 10 in. wide, were made between the arches to afford angles for concealing the vertical joints in the wall. In the 53-ft. arches the flanging of the walls is 9 in., making the ribs 4 ft. wide, but a sheeting of concrete, 18 in. thick, connects the ribs into pairs constituting the barrels, 18 ft. wide.

For architectural effect, the abutment piers at the ends of the main arch are widened to project 3 ft. beyond the general line of the structure, and they provide for refuge bays in the sidewalks of the deck. The twin shafts of these piers are hollow, 6 by 10 ft., giving a thickness of wall of 3 ft., which is increased below the water-tables by the pilasters under the arches. They have a floored chamber under the sidewalk, in order to give access to the electrical conduits and the bases of the light poles. The 10-in. vertical wrought-iron drain pipes from the grate-top inlets in the driveway gutters are secured to the inside wall of these shafts, and pass the drainage out through 16-in. cast-iron pipes built into the body of the pier and outletting on the rocky banks of the creek.

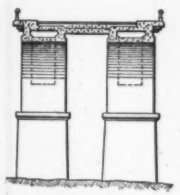
The floor structure consists of transverse rolled-steel I-beams, 6 ft. from center to center, with concrete jack-arches between them. The beams over the 20-ft. middle aisle are 20-in. 65-lb. per ft., and those over the 14-ft. side aisles are 15-in. 42-lb. per ft. The size of the floor members was determined by the assumed concentrated loads adopted as the standard for highway bridges in the City of Philadelphia, which consist of a truck, weighing 40 tons, on four wheels spaced 20 ft. between axles, and 6 ft. gauge, or two street-railway cars side by side, weighing 20 tons on each truck.

The concrete jack-arches can be depended on to distribute an axle load over two or more beams when the axles are more than 6 ft. apart.

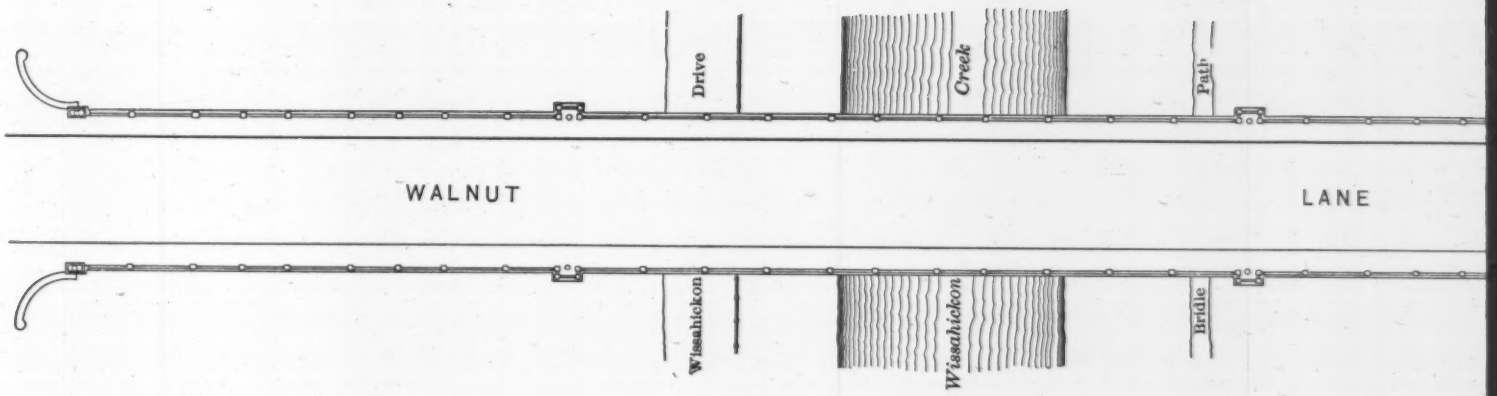
for a beam cannot deflect under its load without throwing some load on its immediate neighbors. The floor concrete is filled monolithically to 3 in. above the tops of the floor-beams. If the spring of the jack-arch be taken at the lower fillet of the beams, and a centroid of pressure be assumed 1½ in. above the tops of the beams, there is constituted for distributing purposes an arch of 12-ft. span, having a rise equal to the depth of the beam, and an effective thickness of 3 in. at the crown. A further assumption is made that the embedded beam will distribute a concentrated wheel load of 10 tons laterally over a space equal to the gauge of the wheels—6 ft. Each transverse foot of floor arch, therefore, is required to sustain 3 333 lb. of live load, and if one-half of this be distributed to the adjacent beams, the horizontal force developed in the distributing arch will give, in the worst case—that of the 15-in. side aisle beams—a unit stress of only 112 lb. per sq. in. Thus it appears unnecessary to compute the beams for more than one-half of an axle load when the axles are 12 ft. or more apart, but the standard weight of the beams used gives value equivalent to five-eighths of the 20-ton axle load at a fiber stress of 15 000 lb., without allowance for the reinforcing value of the encasing concrete. As it is found by experiment, confirming calculations based on conservative assumptions of adhesion of the concrete to the steel, that the embedment of rolled beams in concrete adds one-third to their carrying capacity, the margin possessed by the floor to carry heavier live loads than those specified is so great that the live load may be doubled without stressing the steel beyond common practice. The construction proves to be very rigid under rolling loads.

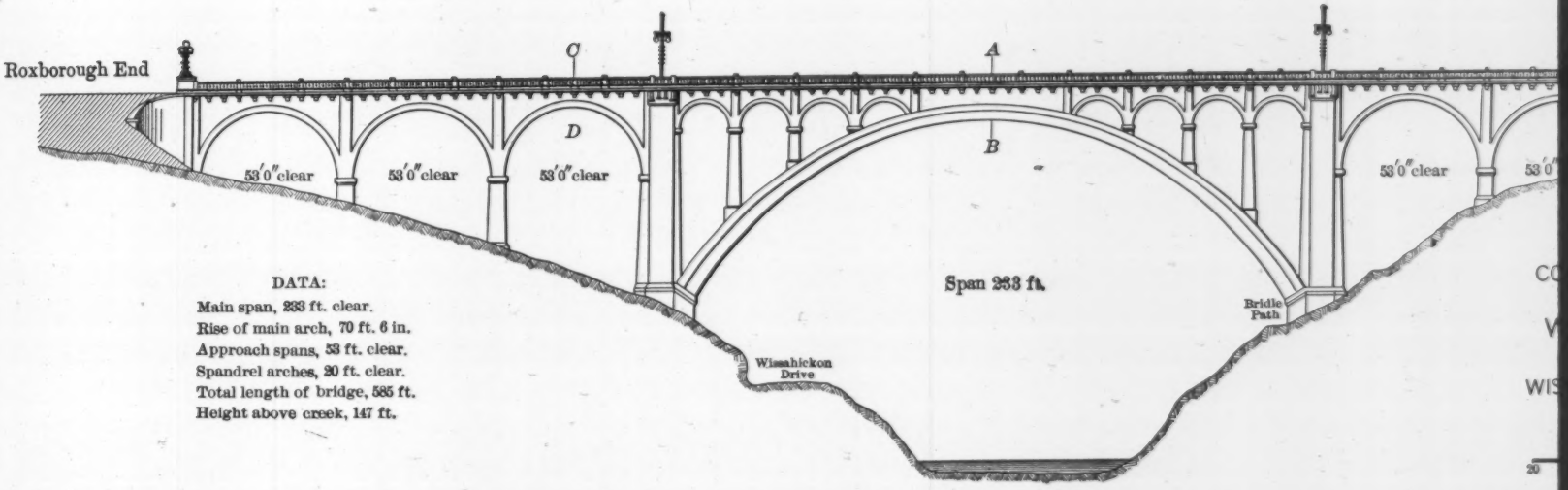
Careful estimates were made of the relative cost of floor construction of this type and of reinforced concrete—concrete beams and slabs reinforced with steel bars. The latter appeared to be the lower in cost by about $4.60 per panel, or less than $1 per lin. ft. of bridge, but at that time experience with construction of that class, for the severe service of heavy street traffic, was limited, and it was felt that the steel beams with independent strength were less of an experiment, and besides, the beams would furnish support for the forms of the floor arches. As these forms were required to be carried high in the air, the support afforded by the beams, and consequently the lesser cost of scaffolding, doubtless quite offset the higher cost of the material.

The 15-in. beams and the 20-in. beams in line with them were



SECTION *C-D*
LOOKING WEST

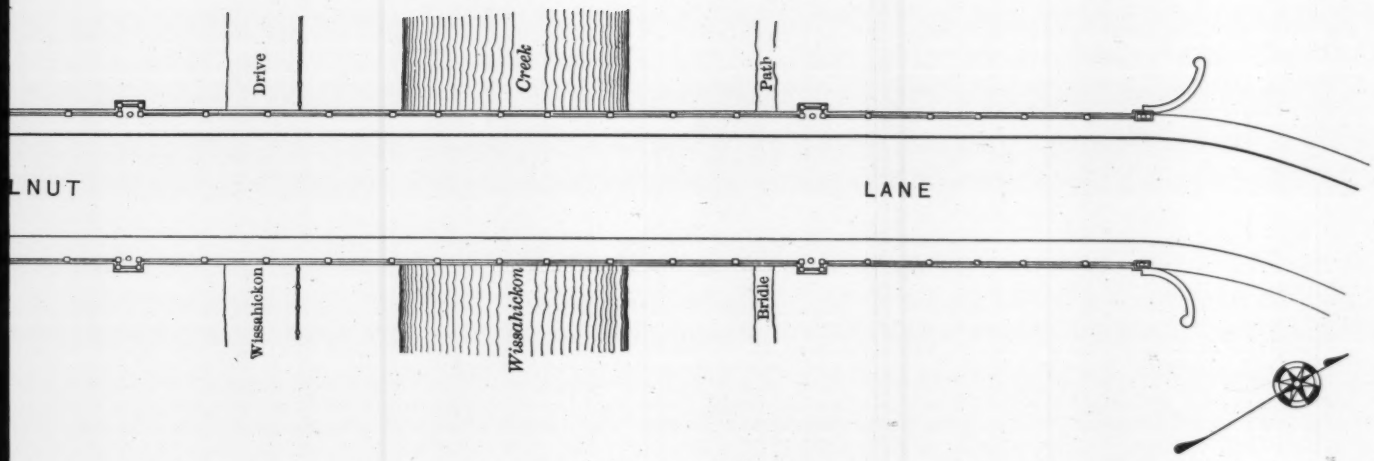


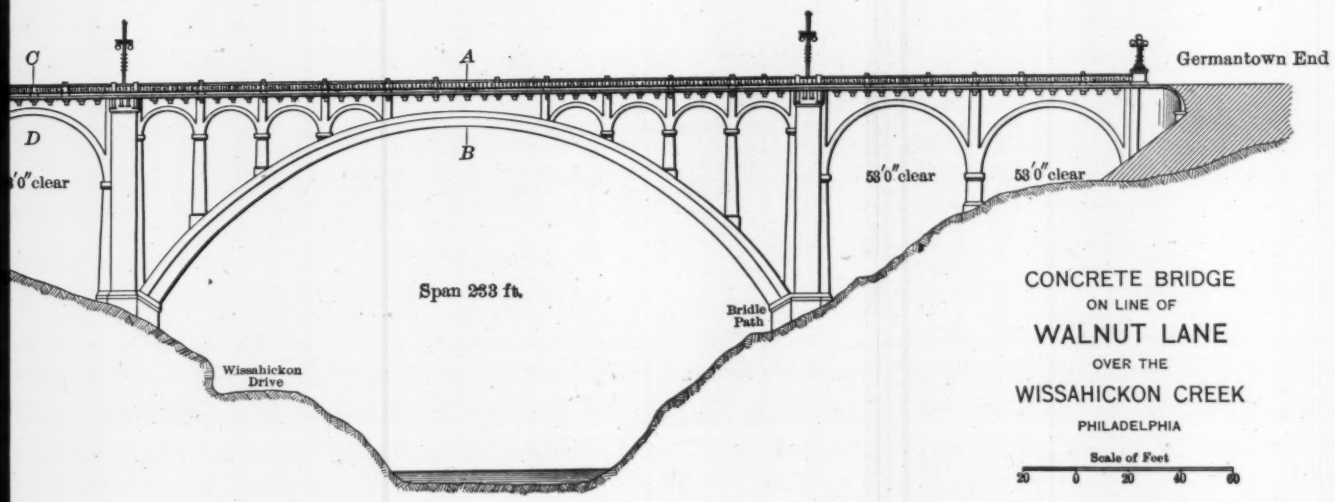


DATA:

Main span, 233 ft. clear.
Rise of main arch, 70 ft. 6 in.
Approach spans, 53 ft. clear.
Spandrel arches, 20 ft. clear.
Total length of bridge, 585 ft.
Height above creek, 147 ft.

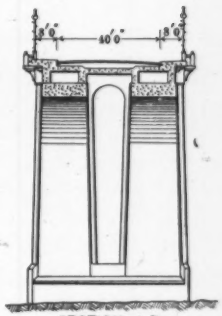
PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.





CONCRETE BRIDGE
ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
PHILADELPHIA

Scale of Feet
20 0 20 40 60



SECTION *A-B*
LOOKING WEST

firmly connected by heavy angle irons with four $\frac{15}{16}$-in. turned bolts, in order to constitute a tie between the two ribs of the arch, and prevent any possibility of a movement apart and split of the floor.

All the beams are completely encased in concrete, which is 3 in. thick over the top surface and around the bottom flanges. At the latter point, $\frac{3}{16}$-in. wire loops, 3 in. apart, are used to prevent the concrete from loosening.

The floor over the middle aisle is depressed to permit the use of wooden ties and cinder ballast for two street-railway tracks, and to afford also, if it should ever be required, depth for an underground trolley conduit.

The whole of the driveway floor surface below the paving is waterproofed with a coating of coal-tar roofing pitch, from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick.

The depression is filled with cinders, over which is laid the 6-in. concrete paving base. The driveway, 40 ft. wide between curbs, is paved with asphalt, and the two sidewalks, 8 ft. 3 in. wide, are granolithic with steel-edged curbs.

The 2½-ft. longitudinal walls are connected at intervals over the piers and ribs of the approach arches, and over the ribs of the main arch, by 24-in. cross-walls, which serve as braces for the walls and as diaphragms or transverse bracing to transmit to the arch ribs the wind forces on the floor. The floor is separated by joints into sections to localize the contraction, and the interruption of its continuity prevents its action as a horizontal beam to transmit the wind stress to the main piers. Each section, therefore, is required to transmit its horizontal forces to the arch ribs. To provide the requisite stability in the long slender ribs of the main arch, they are flared 1 ft. 9 in. on each face, from the crown to the spring, increasing the width from 18 ft. at the crown to 21 ft. 6 in. at the spring. This flaring corresponds to a vertical batter on each face of 1 in 40 in a projected cross-section, and the edges of the ribs, therefore, are in planes parallel to and almost coinciding with the battered faces of the piers. It also causes a slight but not noticeable warp in the face surface, the ribs being made rectangular in section at every point in a radial direction. The thickness of the ring is 9 ft. 6 in. at the spring and 5 ft. 6 in. at the crown. The ribs of the approach spans are not flared—the width being 18 ft. throughout—and their faces are plumb.

The transverse walls do not extend over the space between the ribs, the latter being independent of each other, but the shafts of the piers at the ends of the main arch are connected by a transverse arch of 12 ft. clear span and 10 ft. wide, having embedded in it twelve 1-in. square steel bars diagonally crossed to act as transverse bracing for the shafts. The transverse walls on the ribs are 2 ft. thick throughout the bridge, except those over the west half of the main arch where they were increased in thickness to 2 ft. 4½ in., in order to provide sufficient weight to make the dead load on the arch symmetrical by counterbalancing the heavier load on the east half caused by the greater height of the spandrel piers due to the ascending grade over the bridge.

The abutments were especially designed to economize material and at the same time avoid too utilitarian an appearance. The thrust of the end arch ribs is utilized to balance the earth pressure; and the curvature of the wings, besides embellishing the structure, has the effect of broadening the foundation and thus giving the equivalent of a much thicker wall to resist overturning. This effect is increased by the construction of a buttress, 4 ft. thick, at the extreme end of the wing in a direction radial to the curve, or at an angle of 45° with the axis of the bridge. At the junction of the wings and the front wall, steel rods are embedded for the double purpose of giving cantilever value to the projecting wings, and preventing vertical cracks in the sharp re-entrant angles of the corners. Some of these rods are continued in a straight line past the middle of the curve of the wing and within 6 in. of the surface, in order to give, if it should be required, beam value to the wing between the abutment wall and the buttress at the end of the wing.

The east abutment is 64 ft. high, from foundation to coping, about 38 ft. of this being above the natural surface of the ground in front of it. The south wing of this abutment is built around a 30-in. water main, a hole 6 ft. in diameter being made in the wing to give free access to all sides of the pipe for repairs. The foundations of all the piers were required to be constructed over this pipe, and in each case an arch was formed over it to give 18 in. clear space around it, steel rods being embedded over the openings to resist any tendency to crack. The foundation of Pier 4 was extended because the pipe arch was near the end of the pier.

All piers, abutments, and wings, were founded on solid rock, the

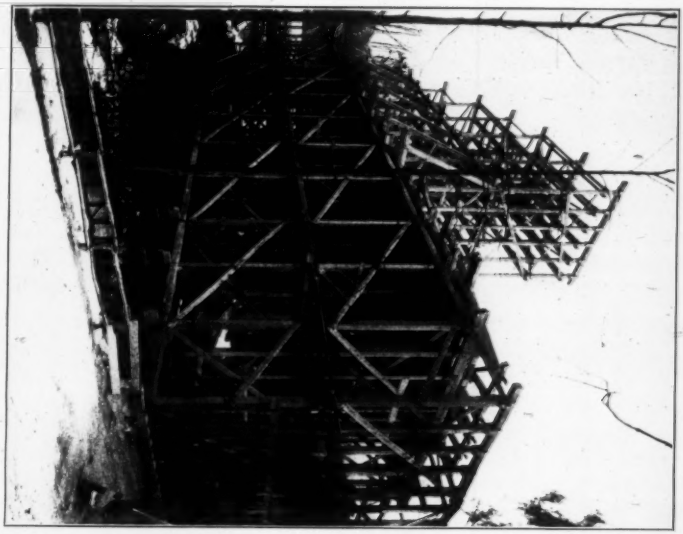


FIG. 1.—FALSEWORK, WALNUT LANE BRIDGE.

FIG. 2.—MAIN ARCH CENTERING, WALNUT LANE BRIDGE.

surface of which was found to be extremely rugged, requiring no stepping. In the foundations of each of the main arch piers two drill holes were sunk 8 ft. below the bottom to make sure of the solidity of the rock, and no crevices were found. The rock is a mica schist.

The open character of the spandrels on the main arch, with their numerous vertical joints, presented a condition quite different from that commonly found with solid spandrels of rigid walls and earth filling, where the load is distributed and the tendency to buck up is met with concentrated resistance. Here, the loads, except immediately over the crown, are all concentrated, and there is no stiffness in either walls or floor to concentrate resistance to buckling. Each panel delivers its own individual load, dead and live, at the same point always, and resists to only the same amount. The long ring rib, therefore, acts as a column, square-ended, throated in the middle, subject to varying eccentric loads, and made up of blocks accurately fitted together but not adhering, and, consequently, it is dependent for its stability entirely on its sufficiency of thickness to accommodate the varying center of pressure. Therefore, it became specially important to determine the extreme eccentricity of the pressure line by trying every possible condition of loading, both concentrated and distributed. The effects of six different conditions are shown by the stress diagram, Plate XXVIII. The shape adopted for the arch is an intermediate between the extremes of eccentricity, coinciding very nearly with the line of dead or constant loading, with a slight arbitrary allowance at the spring for possible initial stress in the intrados from any shrinkage or settlement which might take place after keying, or from being keyed at a temperature above mean, as seemed likely to be the case. This shape was obtained by compounding segmental curves on three centers for each surface, as shown on Plates XVII and XXVIII.

The arch was made square-ended, and continuous at the crown, in preference to hinging it. While pivotal bearings at the three points would have eliminated temperature stresses and reduced the likelihood of initial stresses from falsework settlement, the cost would have been greater, and far less graceful lines would have resulted. The line of center pressure, if confined to pivots at springs and crown, would depart far from the mid-line of the ring at the quarters and require considerably greater thickness there than is needed with the flat-end

bearing. The continuous taper in thickness as well as width of rib, from the abutments to the crown, is an important element of beauty in the design. In further pursuit of lightness in the appearance of the ring, a simple moulding was constructed on the outer face of each rib, tapering in width in proportion to the thickness of the ring.

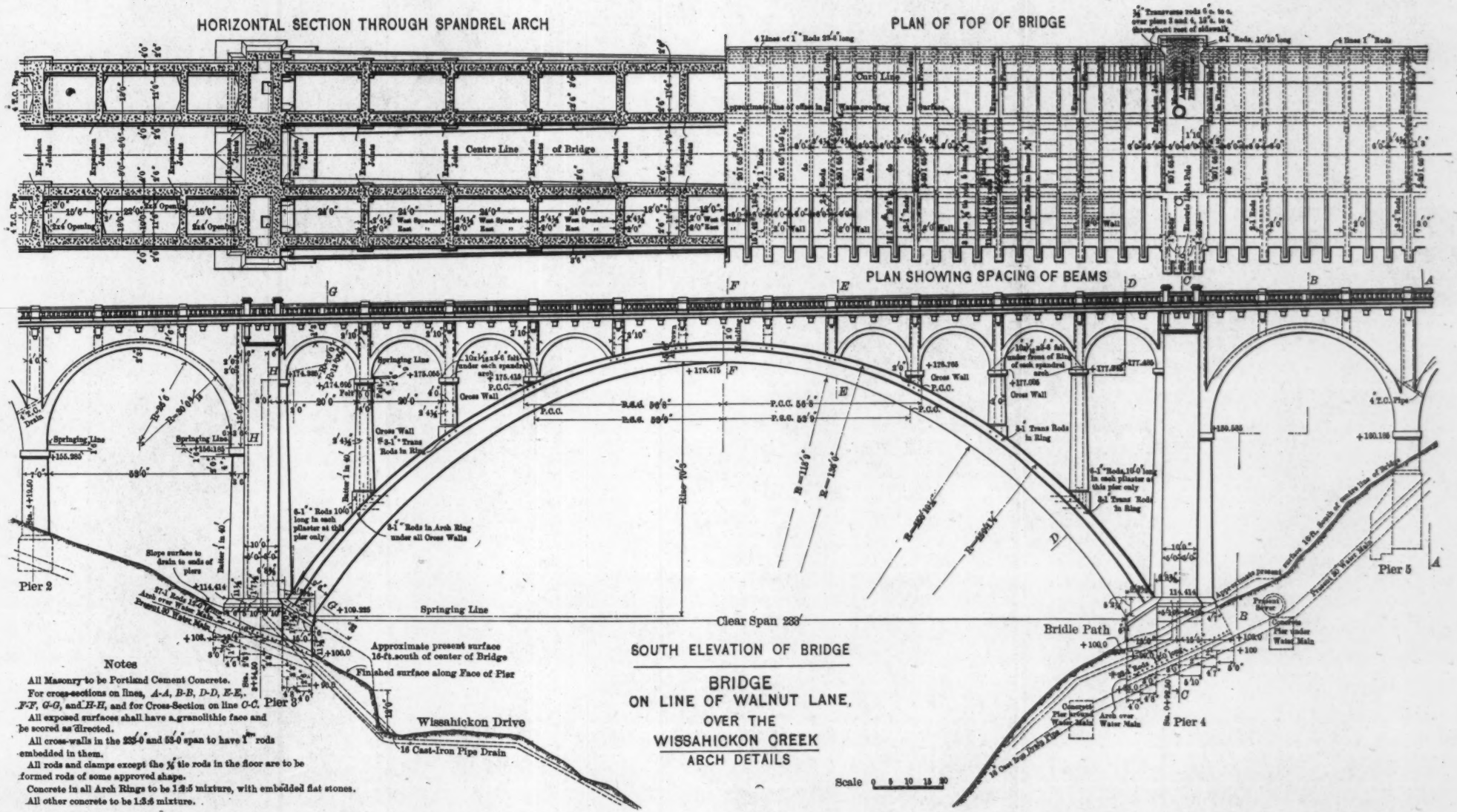
The span of the arch, from center to center of bearings on the skewbacks, is 240 ft., and the rise of the mid-line is 70.5 ft.

The maximum fiber stress appears to be at the spring, where there is the greatest theoretical eccentricity, because of the arbitrary allowance made for initial stress. This maximum is 380 lb. per sq. in., and the average at the same point is 240 lb.

The temperature stresses were computed in two ways, the results corresponding fairly. One method was by using the ordinary deflection formula for beams, and the other was by calculating the increase and decrease in length of extrados and intrados on the curvature resulting from the change in length of the ring and its confinement at the ends, the changes in length of fiber representing stress assumed to be in proportion to a modulus of elasticity of 2 000 000 in both compression and tension. Computations were based on a change of the general temperature of the arch of 40° Fahr., either way from the temperature at keying, which will change the length of the arch 1 in. and cause a change of 1 in. in the elevation of the crown. A rise increases the curvature at the crown, shortening its radius, and—because of the fixed ends—decreases the curvature of the haunches, lengthening the radius. This produces compression of the intrados at the crown—maximum at the middle—and of the extrados at the haunches—maximum at the spring—the effect being zero at the point of contra-flexure. This point was assumed to be at the *K* line of the stress diagram, Plate XXVIII, which was computed by the Cain method, 82 ft. from the spring, and the substantial correctness of the locality was demonstrated by the close agreement of the bending stresses in the several divisions, the deformation of the arch being necessarily an automatic adjustment of the shape to produce uniformity of fiber stress in bending.

Measurements of the changes in elevation of the crown since the completion of the arch show a total difference between the extremes of winter and summer of $1\frac{1}{2}$ in. which is equivalent to a change in length of arch ring of $1\frac{1}{2}$ in. and a change in general temperature

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



SOUTH ELEVATION OF BRIDGE

BRIDGE
ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK
ARCH DETAILS

Notes

All Masonry to be Portland Cement Concrete.

For cross-sections on lines, *A-A*, *B-B*, *D-D*, *E-E*, *F-F*, *G-G*, and *H-H*, and for Cross-Section on line *C-C*.

All exposed surfaces shall have a granolithic face and be scored as directed.

All cross-walls in the 233'-0" and 53'-0" span to have 1" rods embedded in them.

All rods and clamps except the ¾" tie rods in the floor are to be formed rods of some approved shape.

Concrete in all Arch Rings to be 1:2:5 mixture, with embedded flat stones.

All other concrete to be 1:3:6 mixture.

of the concrete of 45° Fahr. The actual temperature of the arch at the time of keying is not known for either rib. It is probable that the first one, because of some heat from crystallization and the atmospheric warmth of the mid-June season, was within 10 or 15° of the highest to be reached, and the second rib, keyed in November, was probably very near the mean temperature.

Some time after the completion of the arch rings, an electric resistance thermometer, of the standard type made by Leeds and Northrup, was embedded in one of the ribs on the mid-line near the spring, and weekly observations were made of the temperature of the heart of the concrete. A diagram showing the results of the changes in temperature and in the elevation of the crown of the arch is shown on Fig. 2. The heavy upper line gives the actual temperature in the arch; the dotted lines show the maximum, minimum, and mean weekly temperatures, as given by the United States Weather Bureau station in Philadelphia. The solid lower lines indicate the elevation of the two arch ribs. It will be observed that the total change of temperature in the center of the arch amounts to 42.8° Fahr., which gives a close check on both the assumptions used in the design, and on the computations of temperature changes made from the change in elevation. The close relation between the various curves in the diagram is also evident.

The concrete of the structure was of several classes, but in all classes—foundations, piers, abutments, walls, and balustrades—stones of as large size as conveniently workable were embedded. Some derrick stones were used in the foundations, but did not appear to be economical, because, under the specifications, a sufficient depth of concrete was required to be in place to half embed the stone, and it was found impracticable to get in as large a volume of embedded stone as by the use of one-man size. Besides, the monopoly of a derrick, during the time of placing a large stone, delayed the delivery of material and reduced the yardage of work accomplished. The quantity of stone thus embedded varied in different portions of the work, according to circumstances—the consistence of the concrete, the character of the sand and coarse aggregate, the convenience of location, and the skill of the workmen. Observations of the relative quantities of stone and mixer concrete used have determined that in horizontal courses, such as are laid in piers and walls, the embedded stone constitutes from 30 to 40%

of the total volume of the finished work, depending on the shape of the stone and the efforts of the laborers. The percentage secured in the arch, where all the stones were required to be placed radially, was from 25 to 30. The contractors were encouraged to use as large a percentage as could be embedded and fully flushed in the concrete, and they were of course desirous of using as much as possible, because its cost was less than that of the concrete displaced by it.

The use of this embedded stone is found to be economical in almost all classes of concrete work, even such as is reinforced with steel rods. Of course, care must be taken in reinforced work that stones do not ride upon rods in a manner to permit the formation of voids under the stones. The increase in compressive strength of all classes of concrete obtained by the embedment of large stones, as indicated by the tests of cubes, enhances the value of the material for almost if not quite all classes of work. The stones also very materially increase the shearing strength of the concrete, and generally reduce its cost.

The concrete of the ring of each 53-ft. arch was deposited continuously, was accomplished always in one day or less, and is of the same character and quality as that in the main arch—1:2:5 mixer concrete with flat stone embedded radially. The skewback was formed at the point of meeting of the arch ring with the transverse walls on the piers, the portion of the arch below this—called the umbrella—being constructed as a part of the pier.

The arches of this bridge, as constructed of this composite material, may be regarded as stone arches of a grade of workmanship superior to the ordinary rubble arch, because of the complete bedding of the individual stones. The increased compressive strength obtained by the embedment of the stone is attended, of course, with a considerable increase in the shearing resistance, and this quality is valued in consideration of the forces operating in the arch at the skewback, where eccentricity of direct stress produces actual shearing stress. The main arch ribs are practically columns, with more or less eccentric loading, and with varying and somewhat indefinite intensity of bearing stress at the ends. Concrete prisms or columns, when compressed to destruction, usually fail by shear, the angle approximating 39° with the axis of compression, and a force acting at any angle to the direction of the compression that causes the shearing will constitute a shearing force and hasten the failure. Such an angular force is produced by

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.

FIG. 1.—WASHING THE SURFACE OF THE ARCH.

FIG. 2.—SOUTH RIB, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

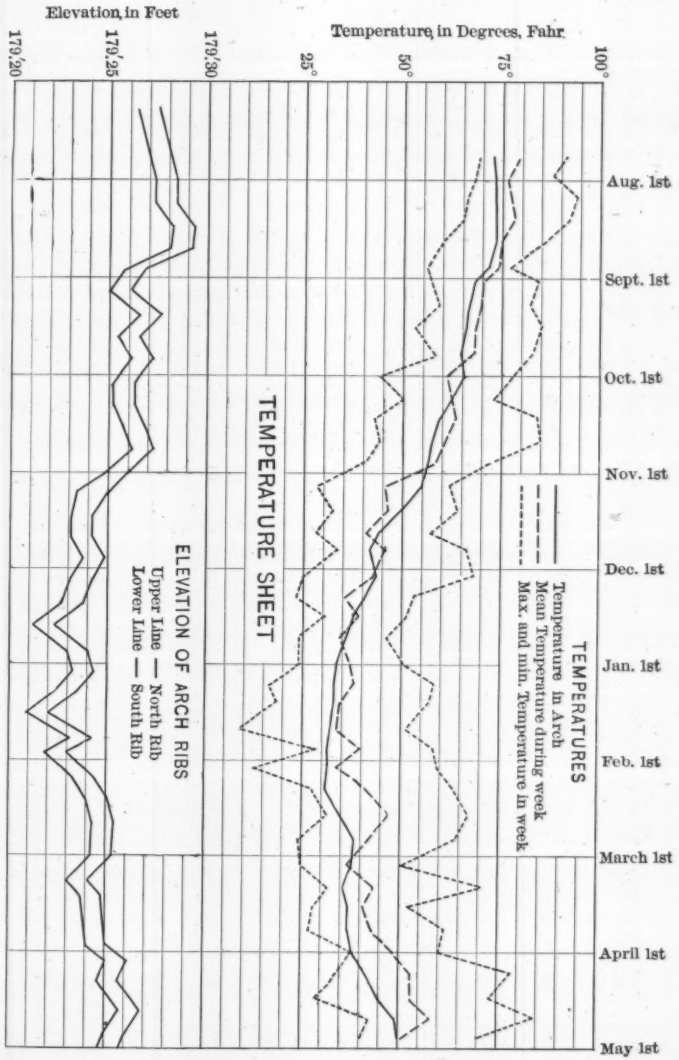

Elevation in Feet
179.20
179.25
179.30
Temperature in Degrees, Fahr.
25°
50°
75°
100°
Aug. 1st
Sept. 1st
Oct. 1st
Nov. 1st
Dec. 1st
Jan. 1st
Feb. 1st
March 1st
April 1st
May 1st
TEMPERATURE SHEET
ELEVATION OF ARCH RIBS
Upper Line — North Rib
Lower Line — South Rib
TEMPERATURES
Temperature in Arch
Mean Temperature during week
Max. and min. Temperature in week

Fig. 2.

eccentricity of direct stress or by increments of loading and own weight. Tests of concrete model arches, made in order to determine questions of design, indicated that it is wise to treat the components of the direct arch stress at the abutment as so much shear acting in a beam. Regarded as such, the vertical component or reaction at the spring of the main arch ribs of this bridge amounts to 108 lb. per sq. in. of arch section in shear at the face of the skewback, which would be high for simple concrete, but is allowable for the hybrid material.

The mix of concrete used in the foundations and in the body of the piers and walls was 1 part Portland cement, 3 parts sand and 6 parts crushed stone. In the arches—the main and approach ribs and floor jack-arches—the mix was 1:2:5. The facing concrete or granolithic mixture of all showing surfaces, and also the body of the balustrade work was 1:2:3. It was all mixed so mushy that the large stone could easily be submerged in it by joggling.

The sand was required to be graded—grains varying from fine to coarse—and a yellow bank sand from New Jersey was accepted and used. Crushed stone was required to be graded in size from ¼ in. up, limited to 1½ in. generally, and to ¾ in. for arch and floor work. After using the 1½-in. size for a time the contractor preferred to use a 1-in. size limit because of the greater facility which it seemed to provide for embedding the large stone. The difficulty, however, with the 1½-in. stone was in obtaining a satisfactory grading—a proper admixture of all sizes from ¼ in. up—the practice of the quarries generally being to separate the 1½-in. stone from the smaller size. If, however, the run of the crusher up to the 1½-in. size can be obtained, the larger size will give equally satisfactory results and perhaps somewhat better concrete. The most thoroughly graded aggregate is found to produce, not only the densest and strongest concrete, but the smoothest working.

Every car lot of cement received was sampled after delivery at the work, and was tested in the municipal laboratory before being used. It was required to meet the standard specifications of the Bureau of Surveys of the City of Philadelphia, which are as follows:

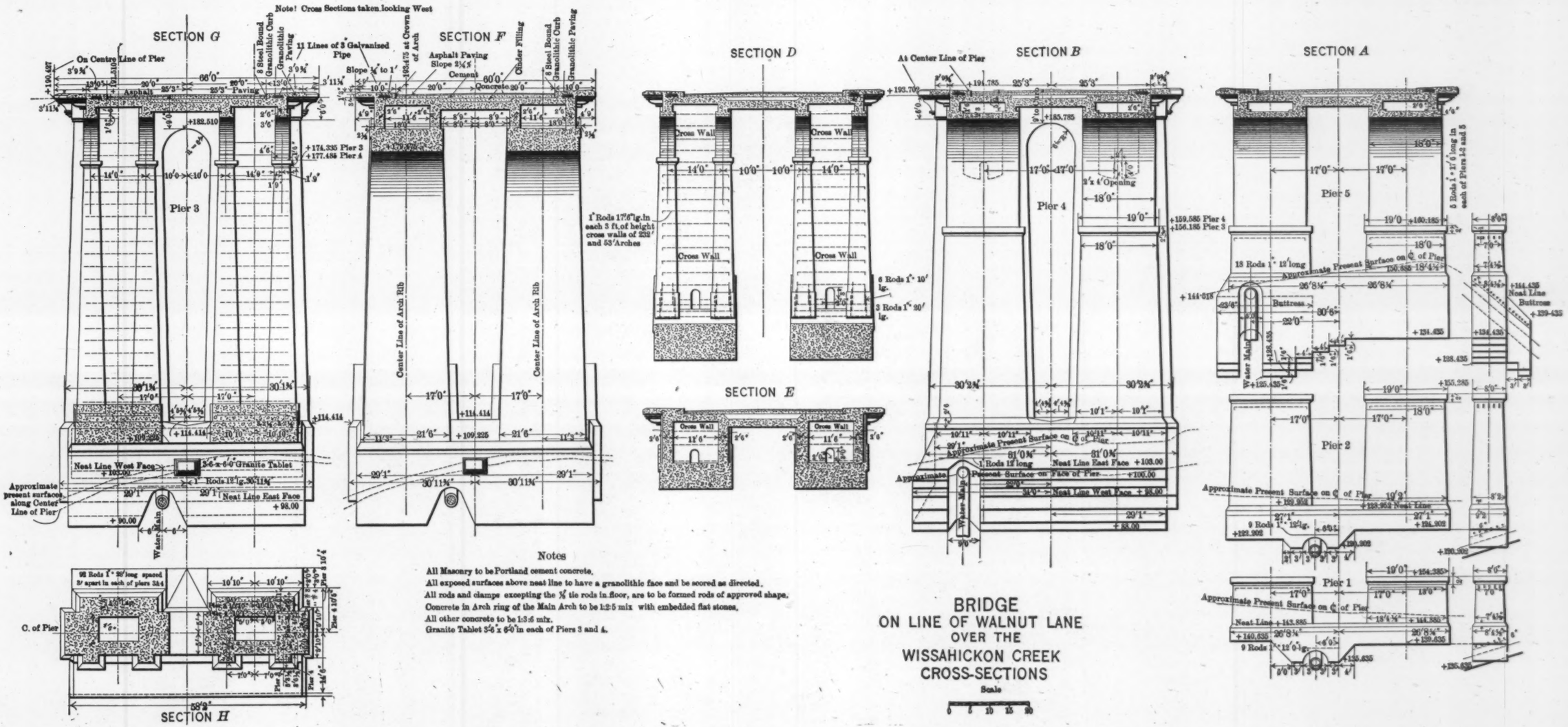
"The acceptance or rejection of a cement to be used shall rest with the Chief Engineer, and will be based on the following requirements:

"Specific gravity not less than 3.1.

"Ultimate tensile strength:

7 days (1 day in air, 6 days in water).....500 lb.

PLATE XIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



Notes

All Masonry to be Portland cement concrete.
All exposed surfaces above neat line to have a granolithic face and be scored as directed.
All rods and clamps excepting the ⅝ tie rods in floor, are to be formed rods of approved shape.
Concrete in Arch ring of the Main Arch to be 1:2:5 mix with embedded flat stones.
All other concrete to be 1:3:6 mix.
Granite Tablet 3'-6" x 6'-0" in each of Piers 3 and 4.

BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
CROSS-SECTIONS

Scale
0 5 10 15 20

28 days (1 day in air, 27 days in water) .....600 lb.
7 days (1 day in air, 6 days in water) (1) one part cement to (3) three parts of standard quartz sand.....170 lb.
28 days (1 day in air, 27 days in water) (1) one part cement to (3) three parts of standard quartz sand.....240 lb.

"Fineness:

Residue on No. 100 sieve not over 8 per cent. by weight.

Residue on No. 200 sieve not over 25 per cent. by weight.

"Set: It shall require at least 20 minutes to develop initial set, and shall develop hard set in not less than 1 hour nor more than 10 hours. These requirements may be modified where the conditions of use make it desirable.

"Constancy of Volume: Pats of cement 3 inches in diameter, one-half inch thick at center, tapering to thin edge, immersed in water after 24 hours in moist air, shall show no signs of cracking, distortion or disintegration. Similar pats in air shall also remain sound and hard. The cement shall pass such accelerated tests as the Chief Engineer may determine.

"Sulphuric anhydride (SO_3); not more than 1.75 per cent.

"Magnesia (MgO); not more than 4 per cent.

"Briquettes for testing shall be 1 sq. in. area of cross-section; sieves shall be of brass wire-cloth having approximately 9 800 and 37 500 meshes per square inch, respectively, the diameter of the wire being 0.0045 and 0.0023, respectively."

The accelerated test used consisted in making a small ball of neat cement paste, allowing it to harden 24 hours in moist air, and then boiling it for 3 hours.

For purposes of record and comparison, specimen 6-in. and 12-in. cubes of concrete, as taken from the mixer, were made by the inspectors at frequent intervals during the progress of the work, and tested in the city's laboratory. Standard briquettes were also made by the inspectors from the mortar extracted from the concrete. Table 1 shows the results of tests of both cubes and briquettes. A number of the cubes had embedded flat stones representing as nearly as practicable the character of the concrete made in the work.

It will be observed that:

(1.) The 12-in. cubes have a strength of about 80% of the 6-in. cubes;

(2.) The 12-in. cubes containing large stones embedded in them average from 25 to 35% stronger than plain cubes;

	1 544	.....	278	2 944	1 903	505	2 317	2 000	485	.....	.....	....	4 928	.....	304	.....	.....	....
	(6)1 325	.....	176	2 029	.....	306	2 391	1 757	379	.....	.....	....	2 917	.....	448	.....	.....	....
	2 228	.....	338	2 678	.....	336	2 436	2 486	466	.....	.....	....	5 164	.....	423	.....	.....	....
	1 792	.....	472	2 172	.....	372	2 401	.....	341	.....	.....	....	4 143	.....	200	.....	.....	....
	2 592	.....	325	1 698	.....	208	3 006	.....	307	.....	.....	....	3 770	.....	213	.....	.....	....
	2 225	.....	332	2 107	.....	373	2 286	.....	305	.....	.....	....	5 369	.....	329	.....	.....	....
	2 015	.....	310	2 636	.....	617	2 406	.....	306	.....	.....	....	4 519	.....	337	.....	.....	....
	2 206	.....	337	2 381	.....	409	2 574	.....	297	.....	.....	....	4 469	.....	382	.....	.....	....
	3 046	.....	475	2 235	.....	319	2 586	.....	420	.....	.....	....	4 167	.....	454	.....	.....	....
	2 415	.....	447	907	.....	123	3 143	.....	476	.....	.....	....	4 723	.....	473	.....	.....	....
	.....	.....	....	2 153	.....	334	3 617	.....	433	.....	.....	....	5 937	.....	485	.....	.....	....
	.....	.....	....	(8)1 300	.....	290	3 188	.....	421	.....	.....	....	4 981	.....	528	.....	.....	....
	.....	.....	....	2 944	.....	645	3 102	.....	162	.....	.....	....	3 619	.....	258	.....	.....	....
	.....	.....	....	3 349	.....	624	3 436	.....	437	.....	.....	....	3 996	.....	649	.....	.....	....
	.....	.....	....	3 834	.....	695	.....	.....	....	.....	.....	....	3 109	.....	514	.....	.....	....
	.....	.....	....	3 147	.....	526	.....	.....	....	.....	.....	....	3 494	.....	503	.....	.....	....
	.....	.....	....	2 583	.....	289	.....	.....	....	.....	.....	....	3 104	.....	477	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	3 065	.....	444	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	2 153	.....	336	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	1 972	.....	275	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	4 021	.....	519	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	4 207	.....	554	.....	.....	....
	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	3 879	.....	422	.....	.....	....
Average.....	1 943	1 748	354	2 648	1 942	420	2 893	2 061	441	3 158	2 288	377	3 657	2 704	393	3 350	.....	410
1:3:6 (2).....	1 784	2 150	336	2 851	2 958	652	.....	3 229	....	.....	2 695	....	.....	3 292	....	.....	.....	....
	.....	2 715	....	.....	3 417	....	.....	3 236	....	.....	2 764	....	.....	3 166	....	.....	.....	....
	.....	2 347	....	.....	2 611	....	.....	2 666	....	.....	3 181	....	.....	2 416	....	.....	.....	....
	.....	(7)1 292	....	.....	2 042	....	.....	2 451	....	.....	2 348	....	.....	3 687	....	.....	.....	....
	.....	2 299	....	.....	2 472	....	.....	2 514	....	.....	2 833	....	.....	.....	....	.....	.....	....
	.....	2 486	....	.....	1 986	....	.....	2 139	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	2 139	....	.....	(8)1 820	....	.....	2 458	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	.....	2 583	....	.....	(3)2 625	....	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	.....	2 931	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
Average (2)..	1 784	2 204	336	2 851	2 536	652	.....	2 665	....	.....	2 764	....	.....	3 140	....	.....	.....	....

NOTES.

(1) = Tensile briquettes of mortar from the concrete.
(2) = Contain flat Conshohocken stone placed flat.
(3) = Flat Conshohocken stone placed vertically.
(4) = Very slow setting, partly fractured in removing from mould.
(5) = Very slow setting.
(6) = Cube spalled, area about 35 sq. in.
(7) = Corner broken, area about 138 sq. in.
(8) = Cube frozen on work.
(9) = Cube spalled, area about 136 sq. in.

TABLE 1.—Tests of Concrete Cubes from Walnut Lane Bridge.

Composition.	Age, 30 days.			Age, 60 days.			Age, 90 days.			Age, 120 days.			Age, 180 days.			Age, 1 year.		
	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)	Compressive strength, lb. per sq. in.		Tensile briquettes. (1)
	6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.		6-in. cubes.	12-in. cubes.	
1:2:2 (2)	2 462	2 111	341	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
1:2:5	1 841	2 055	197	4 236	.....	509	3 426	2 937	377	3 833	2 736	412	5 032	2 625	670	3 890	3 208	432
	2 089	.....	234	3 482	.....	442	4 684	2 875	....	3 702	.....	398	3 916	.....	438	3 664	2 694	435
	3 258	.....	349	4 009	.....	514	4 116	2 181	573	.....	.....	....	.....	.....	....	3 650	.....	299
	3 851	.....	597	4 452	.....	384	4 391	.....	504	.....	.....	....	.....	.....	....	3 001	.....	333
	.....	.....	....	3 150	.....	434	3 209	.....	478	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	3 164	.....	515	3 220	.....	489	.....	.....	....	.....	.....	....	.....	.....	....
	.....	.....	....	2 993	.....	454	3 959	.....	515	.....	.....	....	.....	.....	....	.....	.....	....
Average	2 760	2 055	344	3 641	.....	465	3 858	2 664	489	3 768	2 736	403	4 474	2 625	554	3 526	2 951	375
1:2:5 (2)	2 372	2 111	406	(3)4 111	3 500	843	(3)4 655	2 694	681	.....	3 583	....	4 671	3 805	689	5 072	4 111	455
	2 010	2 861	496	3 512	.....	664	3 918	3 986	690	.....	.....	....	3 760	.....	659	4 375	4 264	477
	1 634	3 174	535	.....	.....	....	.....	4 320	....	.....	.....	....	4 201	.....	611	.....	.....	....
	1 948	.....	646	.....	.....	....	.....	2 806	....	.....	.....	....	.....	.....	...	.....	.....	....
	1 870	.....	369	.....	.....	....	.....	2 764	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 406	.....	375	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	...	.....	.....	....
	2 395	.....	411	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 712	.....	451	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
	2 689	.....	444	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....	.....	.....	....
Average (2)	2 226	2 715	459	3 812	3 500	754	4 287	3 314	686	.....	3 583	....	4 211	3 805	653	4 724	4 188	466
1:3:6	1 091	1 440	385	2 696	2 033	291	3 568	2 097	676	3 613	1 833	238	2 683	3 479	333	3 676	.....	375
	1 993	(5)1 354	394	3 119	2 153	398	3 507	2 472	697	2 157	2 403	362	1 817	2 292	245	3 023	.....	444
	1 754	1 778	392	3 223	2 097	448	3 301	1 777	531	2 414	2 056	443	2 447	2 208	367	.....	.....	....
	2 018	1 916	218	2 704	1 896	387	2 888	1 743	536	2 118	2 861	225	2 453	2 833	273	.....	.....	....
	2 016	1 986	436	3 382	1 799	407	3 679	1 893	670	3 111	.....	339	2 168	2 708	421	.....	.....	....
	2 456	(9)1 694	539	4 017	1 903	481	2 536	2 861	469	2 962	.....	294	3 176	.....	393	.....	.....	....
	(4) 850	1 924	230	2 349	1 687	448	3 097	2 125	491	3 465	.....	457	2 908	.....	290	.....	.....	....
	1 678	1 888	363	3 151	2 083	490	2 829	1 666	554	4 326	.....	576	2 586	.....	236	.....	.....	....
	1 669	.....	253	3 215	1 861	603	2 234	1 916	358	4 256	.....	458	5 074	.....	493	.....	.....	....

(3.) The strength of the concrete shows a fairly regular increase with age;

(4.) The tensile strength of the mortar taken from the concrete is from one-fifth to one-eighth of the compressive strength of the concrete;

(5.) The strength of the 6-months old concrete may be assumed to be at least 2 000 lb. per sq. in.

The crushed stone used in the early part of the work and throughout the construction of the first main arch rib was a very hard black shale of high shearing strength and well graded in size, but was so sharply angular that it was gritty in the concrete and was thought to be less smooth working than limestone. Accordingly, a change was made to crushed limestone, with very satisfactory results. The composition and properties of these stones were as follows:

		Limestone.	Black shale.
Composition:	Passing No. 10 sieve	1.9%	1.5%
	" ¼-in. sieve	15.5%	12.5%
	" ½-" "	44.6%	34.7%
	" ¾-" "	77.6%	70.4%
	" 1-" "	96.8%	94.8%
	Retained on 1-" "	3.2%	5.2%
	Weight per cubic foot	98.9 lb.	90.9 lb.
	Specific gravity	2.84	2.66
	Percentage of voids	44.1	45.1

The embedded stone used throughout the work is known as "Conshohocken" stone—a laminated silicious rock of very high compressive strength—which comes from the quarry in flat shape and with tapering edges, which facilitated the embedment of the stones in a radial direction in the arch rings, and permitted close packing with good bonding in the horizontal courses of the piers, etc.

The showing faces of the concrete throughout the bridge were made with a fine concrete or granolithic mixture, using for all except the balustrades a crushed black shale which had passed a ⅜-in. screen and was cleaned of dust. For the balustrades the crushed stone was granite, ¼-in. size. The mixture used was quite wet, and was deposited against the face forms by pouring from the spout of a coal scuttle, and the body concrete was laid against it in such a manner as to maintain always a coating of the mixture against the forms. As

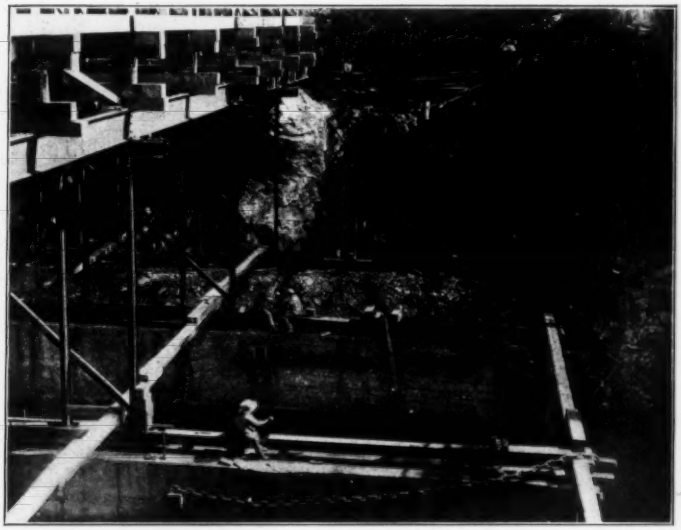


FIG. 1.—MOVING THE CENTERING, WALNUT LANE BRIDGE.

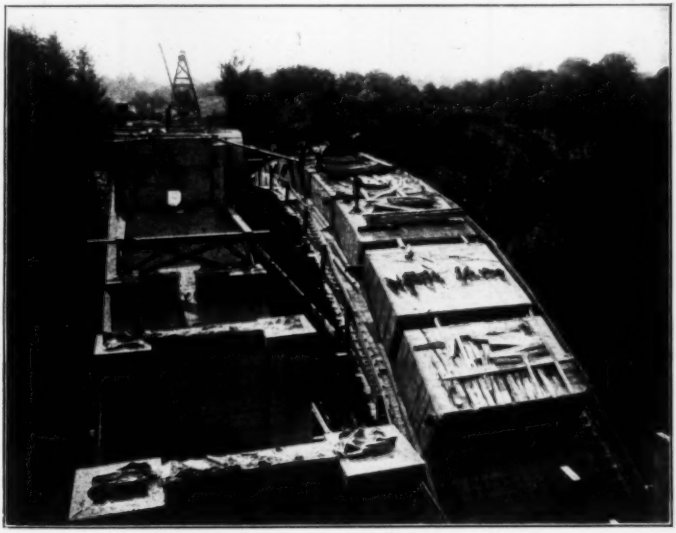


FIG. 2.—VOUSSOIRS OF NORTH RIB, WALNUT LANE BRIDGE.

the mixture and the concrete were both of wet consistency, they readily united to form a monolithic mass. No case has yet been observed of the separation of the facing mixture from the body concrete. The face of the work throughout was finished by removing the forms as soon as practicable after the concrete had set and while it was still friable, and then with brushes and water washing off the surface sufficiently to expose the grit or aggregate of the facing mixture. The forms were generally removed on the day following that on which the concrete was deposited. In cold weather the forms were required to remain from two to three days to permit the concrete to harden sufficiently to maintain itself safely after the removal of the forms.

This process of finishing produces a surface which very closely resembles stone in texture, and, as it removes the cement film which segregates against the forms, all likelihood of surface cracking is eliminated. The tone or color of the surface, of course, depends on the aggregate used in the facing mortar. The black shale grit and the yellow bank sand used for facing the body of this bridge produce the general color and tone of Eastern granite. The mixture used in the construction of the balustrades, which constituted both the body and the face, was made with crushed granite screenings in place of the black shale grit used in the remainder of the work, and a pleasing contrast in tone was thereby obtained.

The use of this fine concrete for surfacing was for the twofold purpose of obtaining a more durable material, it being richer in cement, and a more uniform texture than would result from washing a concrete of coarser aggregate.

To facilitate the removal of the forms for finishing the surface, they were constructed with the uprights or studs set about 8 in. from the face of the work, and the planks of the forms, which were in width equal to the depth of the courses, were secured in place with easily removable cleats tacked to the uprights. On the top and bottom edges of the planks were fastened bead strips, 2½ in. wide, projecting beyond the planks into the concrete to cause imprints. Courses were finished at the apex of the top bead, and, when the forms and planks were removed on the following day to permit the washing of the surface, they were raised one course, so that the bottom bead fitted into the half imprint left by the top bead. The work was thus built up

course by course, in the smaller portions several courses being laid at one operation. No vertical beads were used, and no effort was made to imitate ashlar stone construction.

As the soffits of the arches could not be exposed for washing until the centers were removed, and the surface was then too hard to be removed by a brush, the impression of the lagging was removed by tool dressing. Pneumatic bush-hammers were used from a swinging scaffold, and the work was done before the floor was laid, so that the scaffold could be supported from out-looker timbers on top of the spandrel walls.

The construction of the main arch ribs was by blocks or voussoirs of a size limited to 75 cu. yd. each, always in pairs simultaneously—one on each side—so that the horizontal thrusting effect on the centering would be kept balanced.

The specifications prescribed that each section or block was to be constructed continuously from start to finish without cessation, in order to avoid joints, but in several cases it was found to be impossible to complete the section before dark, and forms were then erected to give to the day's work a radial face. Generally, it was found possible to complete a pair of blocks within daylight of one day—an output of about 150 cu. yd.

The number of large blocks was twenty-two—in eleven pairs—placed in the order of their numbers as shown by Fig. 3, with seventeen small blocks or keys. The first two cast were adjacent to the crown key, thus reversing the usual order of procedure in the construction of masonry arches, although it has been customary to weight down arch centers at the middle with a temporary load to prevent the rising at the crown caused by the thrust of the arch at the haunches. The arrangement was such as to leave seventeen small spaces for keying, the size of these spaces varying from 2 ft. 8 in. for the lower key to 2 ft. 0 in. for the crown key.

The method of construction used—building the voussoirs independently, scattered over the centering, to be joined later by the keys—required extreme care in the building of the forms, but the work was done very accurately, and all the edges of the ribs are in good line, and are entirely without kinks or waves.

The purpose of this method of construction was to distribute the load over the centering and permit the centering to attain its maximum

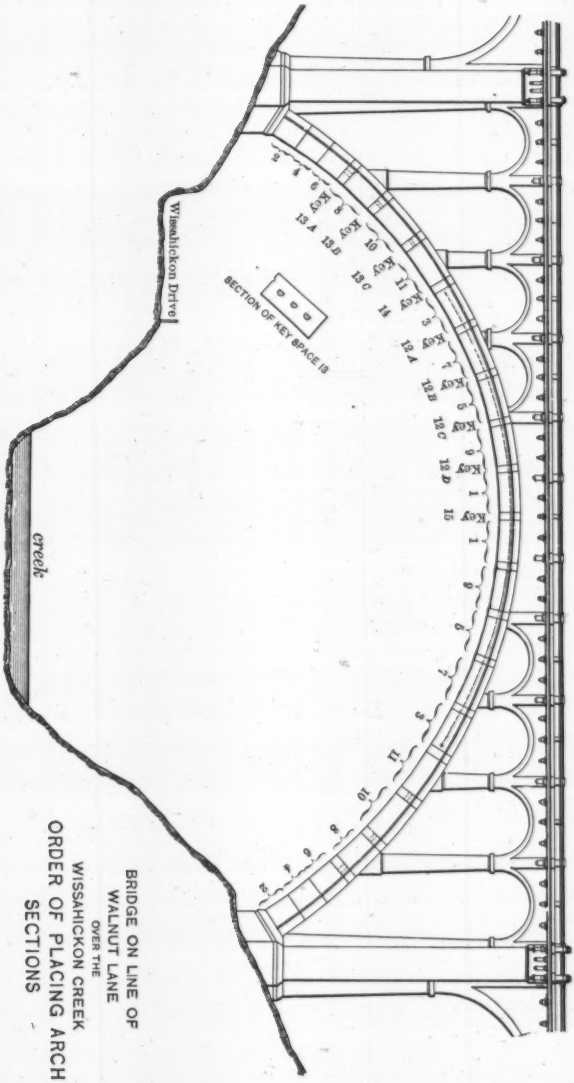
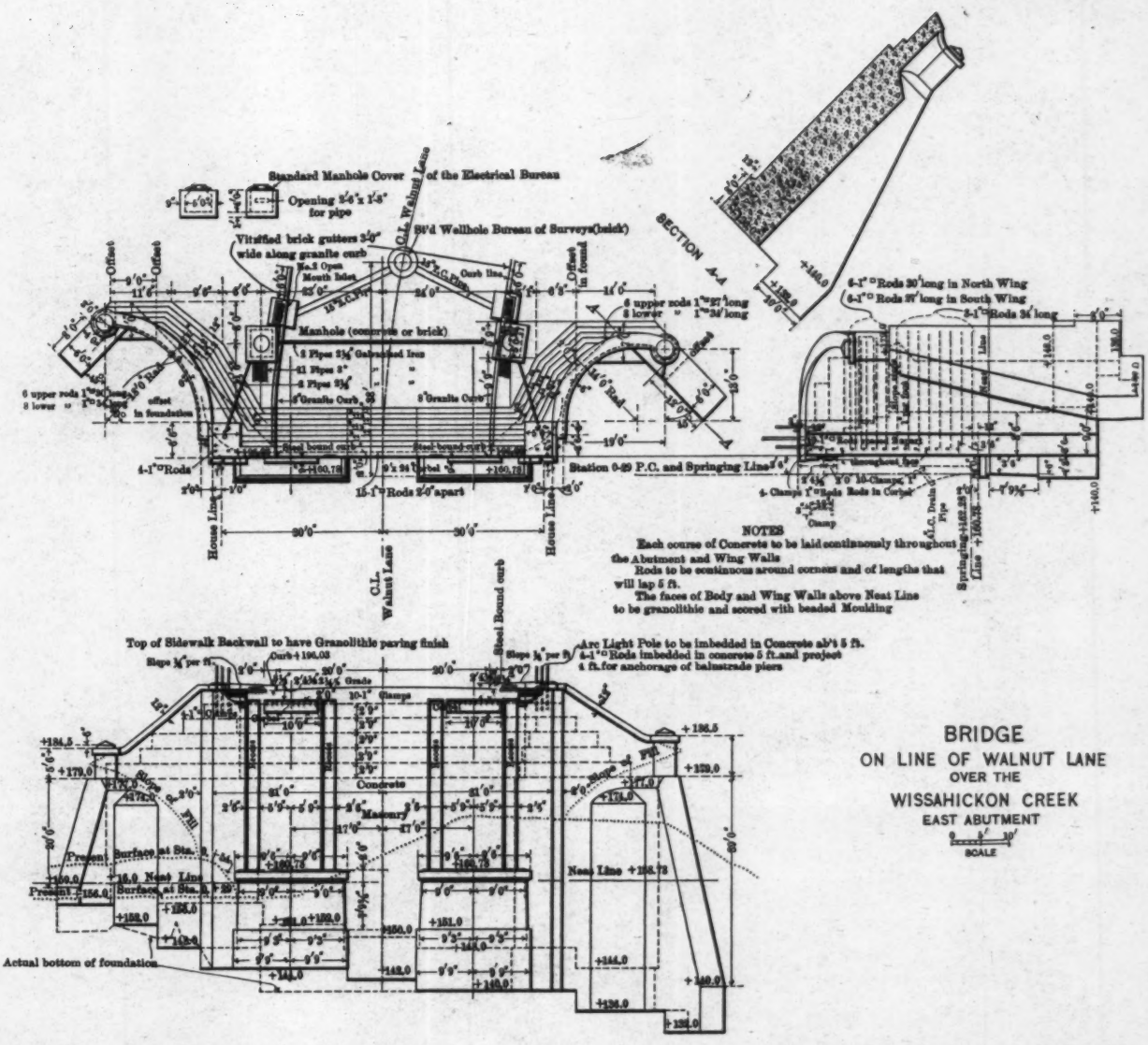

Wissahickon Drive
creek
SECTION OF KEY SPACE 13
Key 1
Key 2
Key 3
Key 4
Key 5
Key 6
Key 7
Key 8
Key 9
Key 10
Key 11
Key 12 A
Key 12 B
Key 12 C
Key 12 D
Key 13 A
Key 13 B
Key 13 C
14
15
BRIDGE ON LINE OF
WALNUT LANE
OVER THE
WISSAHICKON CREEK
ORDER OF PLACING ARCH
SECTIONS

FIG. 3.

deformation before keying, in order to prevent the occurrence of cracks in the arch, a condition which has generally been experienced in the construction of both stone and concrete arches under the usual method of beginning at the spring and proceeding continuously up to and keying at the crown. The utility of the method was demonstrated long before the completion of the first ring. It proved to be entirely successful, and although each main arch rib has in it forty joints, which are without sufficient adhesion of surfaces to give tensile value, there has not yet been discovered any disposition to open anywhere as the result either of loading or of temperature changes.

Some of the individual voussoir blocks on the steeper portions of the centering were required to be secured against sliding down hill on the lagging. This was accomplished in the case of the third pair of blocks—which were at the quarter points—by means of three 1¼-in., round, upset rods, made up to the required length with turnbuckles, and having 12 by 12 by ⅞-in. plate washers at the extreme ends. These rods pass over the crown at the mid-line of the arch, and Blocks 1, 3, 5, 7, and 9, were cast around them, the end washers being embedded in Block 3. No indication of movement of any of these blocks on the lagging could be found, and the one or two that became stressed must have been called into action by a slight movement of the joists. For the three lower key spaces on each side, the separation of the blocks was maintained by small concrete struts on the mid-line of the arch ring, three in each space—18 by 24 in. in the first, 15 by 20 in. in the second, and 12 by 18 in. in the third. As these struts must be built in with the key, they necessarily form an integral part of the arch, and as they must be subject to compressive stress in resisting the sliding of the sections, they must have, because of this initial stress, somewhat excessive pressure on them after the arch receives its full load. The load to be sustained by these struts was in a large measure dependent on the amount of friction on the lagging. Assuming that, if the blocks should move enough to break the bond of adhesion of the cement to the lagging, there would be no friction, the pressure on the struts would be about 300 lb. per sq. in. of gross area. Any shrinkage of the concrete body of a key would entail the shortening of the struts by compression from arch stress before the main body of the key would be subjected to pressure. A shrinkage of as much as $\frac{1}{32}$ in. in the key would require for the corresponding compression of the struts a

PLATE XXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
EAST ABUTMENT

SCALE

pressure of about 2 000 lb. per sq. in. if the modulus of elasticity of the concrete be 2 000 000 lb. per sq. in. This pressure on a small enclosed area would not cause any injurious effects on the material. The manner in which the keys were constructed, however, practically eliminated shrinkage, being in the nature of underpinning work. The mushy concrete was deposited in shallow layers, and more or less wedge-shaped stones were worked and rammed into it. The only visible effect of shrinkage in any of the keys was at the extreme top of the upper joint, where a very fine hair-line crack was distinguishable. In many not even this could be found because the concrete was banked and dammed up higher than the top edge, on the sink-head principle, this extra body being dressed off after hard set.

The anticipated hinge action of these struts developed as the load on the center increased, and it was demonstrated by their progressive cracking as the successive blocks were cast. The crack in each case opened on the upper or outer side of the struts, extending generally to within 6 in. of the under side, thus reducing the actual area for service to a fraction of what was provided, but it was evidently ample for the actual requirements. No special effort was made to grout up these openings, and any failure of the liquid of the concrete to penetrate and fill them would of course leave some small portion of the arch section unflushed and void, but the area thus lost was trifling, and, moreover, was at the least important part of the arch—the mid-line.

The theory of the construction of this arch, involving the principle of numerous keys and temporary hinges, logically called for a key space at the springing line, but it was omitted there because the satisfactory construction of a key would have been difficult, and the struts necessary for maintaining the space during the construction of the remainder of the arch would have been large enough to interfere somewhat with the proper deposit and compacting of the concrete of the body beneath the struts. The three lower blocks, therefore—Blocks 2, 4, and 6, Fig. 2—were cast in contact, and thus formed practically one section about 25 ft. long, directly on the skewback and with its center of gravity about 4 ft. in front of the face of the abutment. The correctness of the theory referred to—that a key was necessary at the spring—was demonstrated when Block 8 was cast. The pressure on the falsework, at this stage of the construction, caused sufficient yielding of the centering to permit the heavy 25-ft. section

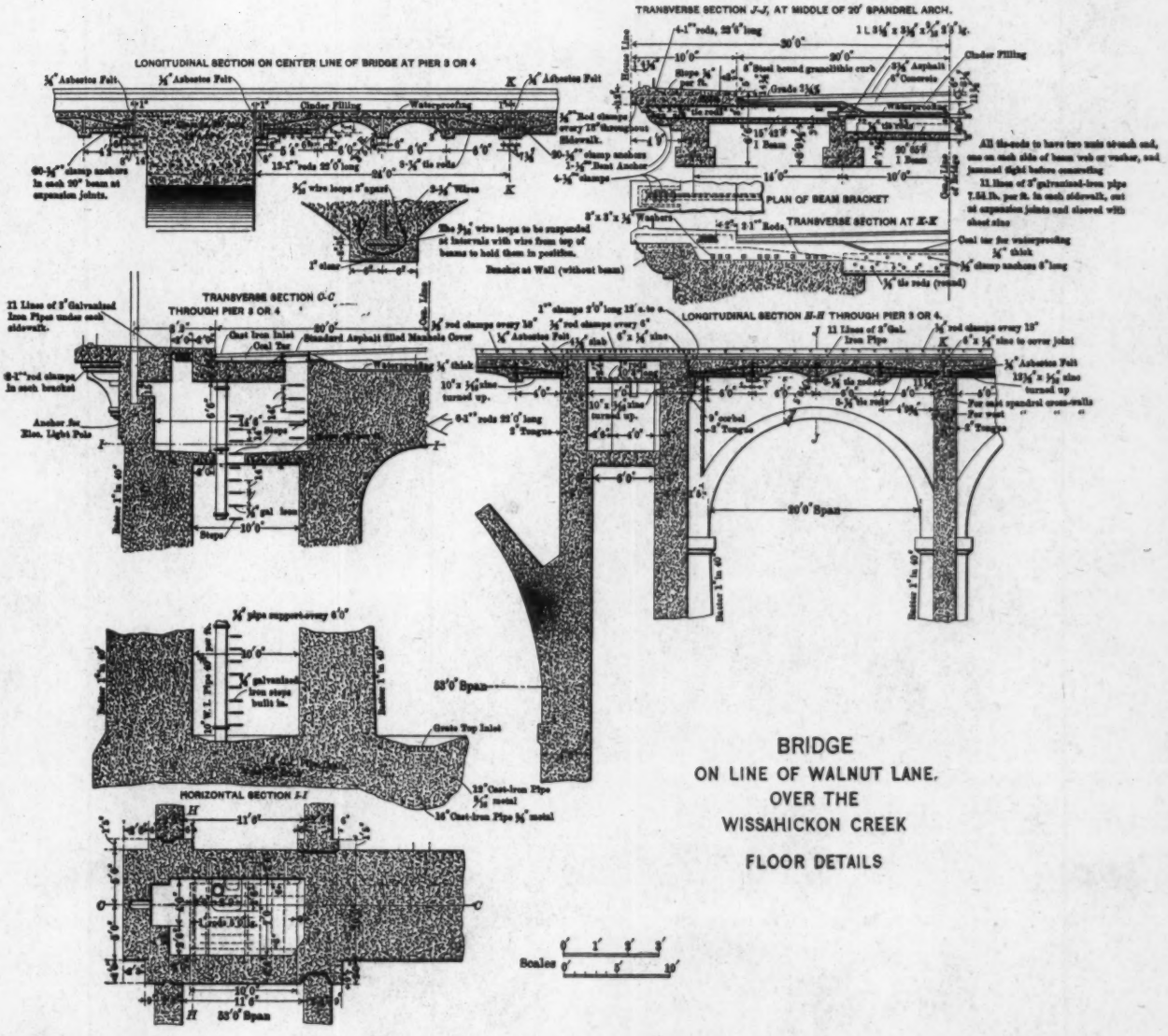
to tilt forward and separate from the skewback at the outer side while bearing on the inner or lower edge. This action occurred at all four of the skewbacks, the cracks thus formed varying in size of opening from $\frac{1}{8}$ in. on one skewback to $\frac{3}{8}$ in. on another at the extrados of the arch, and extending to within about 12 in. of the intrados.

As the construction of the arch on the sections as tilted would have concentrated all the arch pressure on a fraction of the area of the ring at the skewback, it was necessary to push these sections back until their bearing was uniform over the whole width of the skewback. This was accomplished with screw-jacks and wedges acting under the joists and against the horizontal timbers of the falsework, and it is believed that, because of this adjustment, practically all initial stress in the arch ring was avoided. The distribution of the voussoir blocks over the centering, before constructing any of the keys, gave to the falsework almost all its load, and produced the maximum deformation in it before the keys closed the ring. The entire absence of openings in any of the joints between keys and sections, and of any cracks in the sections at any subsequent stage of the erection, is proof of the fact that the line of pressure was always within the middle third of the depth of the ring, for none of the joints have any tensile resistance to opening.

One of the ribs was subjected to an unintentional but interesting test of its stability by inadvertent unsymmetrical loading during the construction of the spandrel walls, which work was done after the centering had been removed. The work of construction over the arch was required to be carried on simultaneously on opposite sides, in order to avoid unsymmetrical loading, but, through a combination of circumstances, the rule was neglected for a part of a day, and a weight of about 100 tons of concrete was deposited in the spandrel walls of one section immediately over one of the quarters without any balancing load on the opposite quarter. As soon as the condition was discovered a careful examination was made of all the joints of the arch, but not the slightest evidence of distortion could be perceived. A pressure diagram, covering this condition, was made subsequently, and the center of pressure was found to be within the middle third of the arch ring. This is accounted for by the great weight of the arch ring itself.

The centering was required to be maintained under the completed

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



BRIDGE
ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK
FLOOR DETAILS

arch ring for at least one month. The first ring was keyed in a season of rising temperature (June 7th), and the wedges of the centering were struck one month later. The second arch was keyed in cool weather (October 17th), and the support was continued for 6 weeks. The stresses in the woodwork were high enough, apparently, to produce progressive deflection, so that, during the time of loading and subsequent seasoning, the falsework compressed sufficiently to lower the crown 3¾ in., this amount being the same for both ribs. In anticipation of this compression, the centering was originally set higher than the plan elevation. During the construction of the keys the woodwork was saturated with water as far as practicable by flowing water upon the bents through holes in the lagging, and the water was shut off as soon as the keys were completed, the purpose being to obtain the advantage of the shrinkage of the wood in drying out to withdraw gradually from the arch some of the support of the centering, and thereby facilitate the striking of the wedges. The utility of this process is doubtful when as high stresses are put in the wood as was done in this case, for it is probable that the softening of the unconfined fibers of the wood by the absorption of water did not operate to increase the height of the centering by the swelling of the wood, as was expected. The action of the wood during the period following the keying of the arch, apparently, was the continued yielding under the load, indicating that the elastic resistance was less than the imposed stress, and, as the deflection of the crown of each of the two main ribs was only ⅛ in. at the time of striking the wedges, it would seem that the elastic resistance of the whole structure of the centering was just sufficient to carry so much of the weight of the arch ring as would deflect the arch ⅛ in.

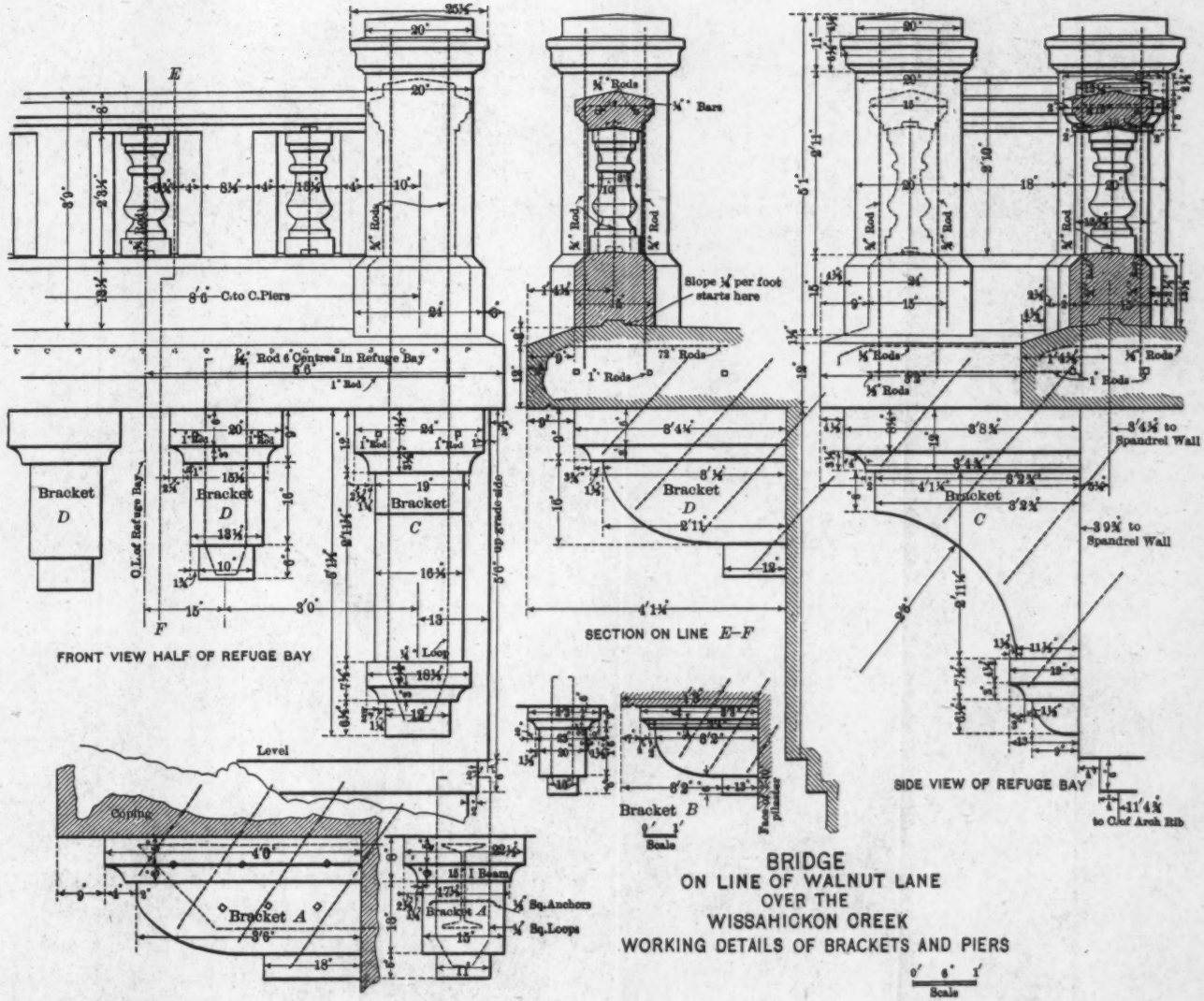
The centering was designed by the contractors. Its general arrangement and details, with stresses and sizes, are shown on Plate XXVII. All bents were supported on concrete piers built on the rock, with the exception of four intermediate ones, which were supported on 24-in. I-beam stringers spanning the creek bays. The lower story of the high bents was made of steel beams and channels braced with angles. The remainder was of yellow pine timber. The working stress in the 10 by 10-in. posts, which had up to 13 diameters of unsupported length, was about 1 000 lb. per sq. in. As the sills, blocking, and wedges, were all 10 in. wide, the area of side-grain bearing was the same as the end grain, and the unit stress, therefore, was the same. The resulting

crushing of the fibers of the yellow pine sills where they bore on the 10-in. oak wedges clearly showed that the stress was beyond the proper working load for side-grain bearing. The extreme fiber stress in bending on joists and on caps of bents was 1 200 lb. per sq. in., and ample strength and stiffness in these members was evident. The steel I-beam stringers were stressed at 16 000 lb. per sq. in., and the resulting deflection, therefore, was normal, and had no effect on the concrete work. The steel I-beam posts of the lower story had a stress of from 5 700 to 8 500 lb. per sq. in., according to the unsupported length between points of attachment of the diagonal bracing. The 6-in. steel rollers under the high bents were loaded, during the erection of the arch, to 3 400 lb. per lin. in., assuming that the rollers were all equally loaded, which is hardly possible. The total weight of the falsework on the rollers of the middle bent was 100 tons.

A peculiar feature of the structure was the method of connecting the curved joists. There were twenty-five lines of these, lapping at each bent and pinned together there with 3-in. round oak treenails. Instead of being notched to bear vertically on the caps of the bents, the joists were left straight and smooth on their underside, and beveled timbers on top of the caps of the bents were dressed to fit under them and receive their pressure. This construction produced horizontal as well as vertical forces to be resisted. In the lower bents the lateral thrust was greater than the vertical because of the steep inclination, and it was resisted by horizontal struts or rangers extending clear across. At the higher bents, where the inclination was flatter, the horizontal effect of the joist bearing was taken up by plank chocks spiked to the joists above and against the beveled timbers.

The designer assumed that the pinning of the joists would cause them to act as an arch and thus carry a considerable portion of the weight of the concrete ribs. It was apparent, however, that they did not act in this way, and also that the lagging did not arch. The length of arc was about 280 ft. and the shrinkage of the lagging in drying out was more than sufficient to accommodate the shortening of the arc in the lowering of the crown by the compression of the centering, and, although the joists were very carefully bored for the treenail pins, the play in the holes and the compression of the fibers was evidently sufficient to prevent any observable arching. The placing of the joists was commenced at the crown and proceeded downward

PLATE XXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



BRIDGE
ON LINE OF WALNUT LANE
OVER THE
WISSAHICKON CREEK
WORKING DETAILS OF BRACKETS AND PIERS

on both sides, the lower joists being fitted into the end spaces, the tendency of the joists to slip down being resisted by their linking over the crown, whereby the chain of joists was in tension before the lowering of the centering began. The proportion of rise to span is such that a change in length of arc produces almost exactly the same amount of change in the elevation of the crown. Therefore, for the deflection of $3\frac{3}{4}$ in. at the crown, the line of joists must have shortened $3\frac{3}{4}$ in., and, as the end bents depressed $\frac{1}{8}$ in. each, and as there were nineteen pin joints, the yield at each joint, by play and by compression of the wood, must have been $\frac{3}{16}$ in. When the centering was taken down, it was found that many of the treenails were permanently bent into a zigzag shape, presumably due to the effort of the linked joist to close on the pins.

It seems probable that a much stiffer and somewhat more economical structure would have resulted from fitting the ends of the joists and abutting them.

An unsatisfactory detail appeared, also, in the high blocking used between the first and second stories, and between the second and third stories. The caps and sills of the bents at these points were made double, to furnish stiffness between the posts for anticipated jacking, and 20 in. of space was then provided for the jacks. This space, therefore, had to be made up in blocking. Between the first and second stories, two 10 by 10 by 20-in. blocks were used, and between the second and third stories one 10 by 10-in. block and two oak wedges 10 in. wide and 5 in. thick at mid-point. There was thus 60 in. of wood bearing across the fiber, made up of six pieces 10 in. wide. Its objectionable nature appeared in the unsymmetrical yielding of the wood under long-continued pressure, causing the pile of blocks to buckle and begin to topple. No actual failure occurred at any point, but it was evident that if the structure had been required to sustain its maximum load much longer some of the piles of blocks would have collapsed. The unsymmetrical yielding seemed to be due, at least in some cases, to an excess of sap wood on one side of the block—the heart of the wood not being in the center of the stick. In some instances the sap wood split away from the heart wood, having swelled more than the heart wood, probably from absorption of water, and then buckled away.

It was observed that in the higher bents the caps became dished,

on account of the greater compression of the interior posts. The concavity was as much as 1½ in., and the soffit of the arch rib at the crown, therefore, is necessarily that much curved, and the arch is that much thicker in the middle than at the faces. This dishing was not perceptible in the sills of the second story, and only slightly in the sills and caps between the second and top story, the transverse bracing probably acting to distribute much of the load to the outer posts.

The action of the falsework indicated that the unit stresses to which the wood was subjected, in side-grain bearing at least, were beyond its elastic resistance, and it is recommended, therefore, that the working stresses of wood in side-grain bearing, even for temporary duty, should be limited to 500 lb. per sq. in., and that the details should be made as compact as possible.

The means provided for adjusting the height of the bents, and for lowering them in striking the center, consisted of oak wedges, 10 in. wide and 20 in. long, tapering from 7 to 3 in. in thickness, placed crosswise of the sills in order to facilitate driving. When the center was lowered the wedges on the end bents at the skewbacks were loosened first by backing them out with sledges. At each of the four points they were found to have only a moderate pressure on them, proving that there was no considerable arch action on either the lagging or the joists. Then the wedges under the middle bent were slackened by backing them only enough to release a portion of the pressure, and successively all the others—working two gangs of men, one toward each end. A number of the wedges were so tightly held that they could not be started without having the weight on them partly taken up on jacks, one or two 15-ton ratchet screw-jacks being used for each tenacious wedge. One wedge refused to budge, even then, and it had to be split out with chisel bars. The evidently unequal loads carried by different posts of the same bent may be accounted for by different resisting values in the posts, and the consequent deflection and concentration of load by the caps of the bents. After all the wedges had been partially backed, the operation was repeated from the middle bent to the ends, and all bents were lowered several inches clear of the arch. The longitudinal bracing of the bents acted as a truss, and carried the middle bents when their wedges were dropped, thus preventing any bents from lowering much below their immediate neighbors. Notwithstanding the apparently heavy pressure on some of the wedges,

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.

FIG. 1.—END OF CENTERING PARTLY MOVED, WALNUT LANE BRIDGE.

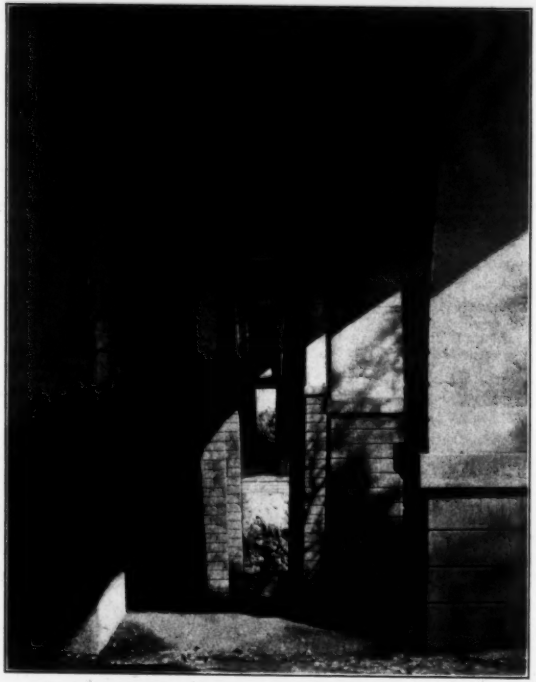


FIG. 2.—VIEW BETWEEN RIBS, WALNUT LANE BRIDGE.

it was clear that, at the time of striking, the centering was carrying only a fraction of the weight of the concrete arch, for the deflection of the latter, upon the withdrawal of the support of the centering, amounted to only $\frac{1}{8}$ in. at the crown and $\frac{1}{16}$ in. at each quarter. This was the case in both ribs. The deflection was measured by three observers, each with an engineer's level, three fixed targets being used —one on the crown and one on each quarter.

One of the reasons for the economy found in the double-rib type of arch was the practicability of building one rib at a time, requiring only enough centering for one rib. Therefore the centering for the main arch was made for a single rib, while the falsework piers were made long enough for the two, and, after the first rib was swung, the whole center—with the exception of the end bents, which were moved independently—was moved over as a unit into position for the second rib. This was accomplished by using rollers under the bents, and jacks pushing on the sills (Fig. 1, Plate XX). The steel bents had been erected on 6-in. solid steel rollers, and 3-in. wood rollers were inserted under the low bents. The jacks heeled against blocks chained to wooden spurs built into the piers for the purpose. The jacks were all actuated simultaneously, and a uniform speed of movement was maintained in order to prevent any skewing of the structure. This operation occupied less than three days.

The centering for the 53-ft. arches was an **A**-frame truss with a collar tie at mid-height, but no tie at the heels; the thrust, therefore, was sustained by the piers. Computation of the dead weight resistance of the piers to overturning indicated that in two cases it was scarcely equal to the overturning moment, and that prudence would dictate the precaution of ties at the heels of the ribs. They were not used, however, and the construction was accomplished without any visible effect of thrust upon the piers. The dowelling of the courses of the concrete masonry of the piers, which was the result of finishing every course with the embedded stone projecting above the surface, is doubtless to be credited with increasing the resisting moment of the piers by giving tensile value to the joints between the courses.

The vertical support of the trusses of the 53-ft. arch centering was obtained on the 6-in. projection of the coping, supplemented with a wooden trestle bent from the ground up in front of each of the low piers, and, in the case of the high piers, a timber corbel bolted to the

face of the concrete. The bolts for the purpose of supporting the form timbers, wherever used throughout the work, were ¾ in. in diameter, and built into the concrete; and the nuts, 4 in. square and ⅝ in. thick, were embedded from 6 to 8 in. As a rule, they were loosened slightly by unscrewing them one turn within a day or two after placing, in order to facilitate their ultimate removal, which was generally accomplished with little trouble. A few, however, were found to be gripped so tightly by the concrete that they were twisted off in the effort to unscrew them from the nuts, and had to be cut off with a chisel. This was done 1 in. or more below the surface, in order to permit a sufficient body of pointing to prevent the metal from rusting and staining the face. All holes left by these bolts were plugged with granolithic mortar and washed to match the face work.

The centers for the 20-ft. spandrel arches were built up and the lagging was nailed on before placing them in position, each center being handled and set as a unit. These were supported on the projecting caps of the piers.

The centering for the floor arches was suspended from the embedded steel beams with ¾-in. loop hangers or saddles over the tops of the beams, blocked with wood high enough to facilitate the removal of the block and the pulling out of the loop after the concrete had become self-sustaining. The centering was removed as early as was deemed safe, in order to get the hangers out with the least difficulty—under favorable conditions for setting, it was done within three days. A few of the hangers were held by the concrete so tightly that they could not be drawn, and were permitted to remain. The removal of the jack-arch centering was accomplished with a light scaffold suspended by single ropes through small holes cored in the floor concrete at suitable intervals. These holes, as well as those left by the withdrawal of the hangers, were not plugged, but were left open for the permanent drainage of any water which might get through the paving, but the spaces left by the removal of the wood blocks were filled with concrete.

All material was handled by a cableway of 800 ft. span, supplemented at the piers and on the floor by swinging boom derricks. The concrete mixing plant was under the cable.

The provisions for temperature changes throughout the structure have thus far proved to be effective in preventing cracking or spalling. The system adopted consisted in general of dividing the structure into

FIG. 1.—FLOOR SYSTEM, WALNUT LANE BRIDGE.

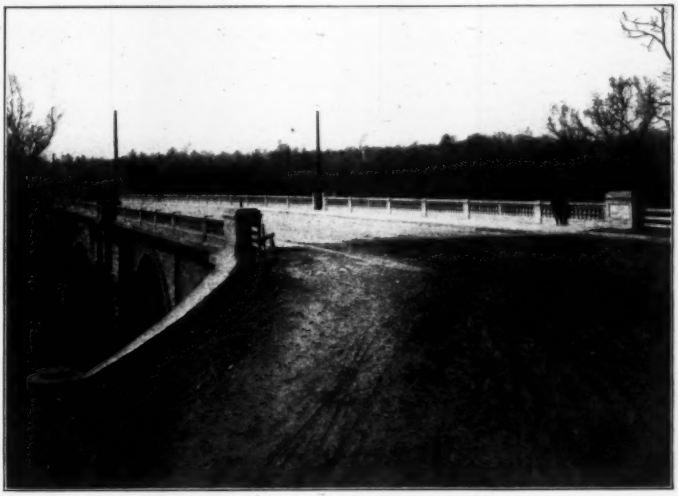


FIG. 2.—DECK VIEW, WALNUT LANE BRIDGE.

sections, all the joints being locked with groove and tongue, and cushioned with a layer of soft asbestos felt. Steel reinforcement was not used anywhere to resist temperature effects. It has been observed in concrete structural work that unequal expansion of parts has caused rupture or spalling. Some parts—such as the rail of a parapet—may have less mass of body, and be also more completely exposed to the atmosphere, than other parts—such as the covered arch ring—therefore responding more quickly to changes in atmospheric temperature. They will then expand before the other parts, and something must yield. It has been found that, at points where such action is likely to develop, a sheet of soft mineral-wool felt, $\frac{1}{16}$ or $\frac{1}{8}$ in. thick, according to circumstances, will afford an elastic cushion which, by compressing, absorbs expansion, and, by swelling, follows contraction of the concrete, thus keeping out dirt and minimizing the appearance of the opening. Care must be taken, of course, to have the felt complete over the whole abutting surface of the joint. If any portion, particularly an edge of the joint surface, has the concrete in contact it may spall in expanding.

The floor structure, being of jack-arches, required staying at the joints, in order to take up the thrust of the arches. This was done with tie-rods. It required also double floor-beams at each joint in the middle aisle, independent of each other and separately encased in concrete. The joints are located so that in the side aisles they are over the cross-walls. Any movement in the floor structure at its joints necessarily involves a little sliding on bearings. Wherever this action is likely to occur, the seat of the sliding side was made with a sheet of zinc, $\frac{1}{16}$ in. thick, flanged up at the back to prevent its dislodgment. Its smooth surface facilitates the sliding, and it does not rust. The same device was used at the base of such of the balustrade posts or piers as occur at floor joints. The joint was necessarily made on the mid-line of the supporting bracket and wall, but it was not desirable to split the pier, so the half of the pier which had to be built over the sliding side was bedded on sheet zinc, and the joint at the side of the pier in the balustrade completed the provision for movement.

The joints in spandrel walls were made about 24 ft. apart over the main arch—one alongside each cross-wall—and in the 53-ft. arches, because of the greater depth and consequent rigidity of the walls, the joints are only about 18 ft. apart. In the floor the joints are as much

as 60 ft. apart, and the movement has not been enough at any one of them to develop a crack in the paving of the roadway, which is continuous over the joints. The joints in the sidewalks have the felt cushion.

Provision was also made for hinge action in the spandrel arches, due to the change of relative elevation of their springing points with the rise or fall of the crown of the main arch. The change of elevation of the crown from variation in temperature is $1\frac{1}{8}$ in. The spandrel piers, therefore, move up and down, each an amount proportionate to its distance from the middle of the span. Thus, the one nearest the end rises approximately one-fifth of $1\frac{1}{8}$ in., the next one two-fifths of $1\frac{1}{8}$ in., and the third three-fifths, etc. As this movement is necessarily vertical, the effect on the small arches is a bending with rotation about their springing points. To minimize the tendency of this action to spall off the concrete of the intrados at the spring, or to open cracks higher up, a sheet of asbestos felt was built in the whole width of the rib, 3 ft. 6 in., extending in from the soffit 9 in. Of course, this reduces the end bearing and increases the clear span of these arches, but they are of more than ample size for their duty.

Steel-rod reinforcement was used in general only at points where it was thought that the concrete was likely to crack on account of shrinkage or other indeterminate stresses. At the few points where load conditions produce tensile stresses, the amount of tension is so low that the flat embedded stone would furnish ample value to carry them, several tensile tests of the stone used showing strengths of from 1 000 to 1 200 lb. per sq. in. These points are the sidewalk brackets which are opposite cross-walls where steel beams were not used in the side aisles of the floor, and the 4 ft. 9-in. overhanging floor slab which is supported on the sidewalk brackets. In the former, two 1-in. square rods were embedded, extending into the concrete of the middle aisle for transverse tie purposes, and in the floor slab four 1-in. square rods were embedded in a longitudinal direction. In addition to these, transverse rods, $\frac{1}{2}$ in. square and 18 in. apart, were embedded in the sidewalks to prevent the possible tendency to crack longitudinally at the point of overhang—the outside line of the spandrel wall. For similar reasons, rods were embedded transversely in the spandrel piers on the main arch, where the cross-web walls are much less in thickness than the pilasters which they connect; also, in the high hollow shafts of the

FIG. 1.—BOTH RIBS, WALNUT LANE BRIDGE, AFTER REMOVAL OF CENTERING.

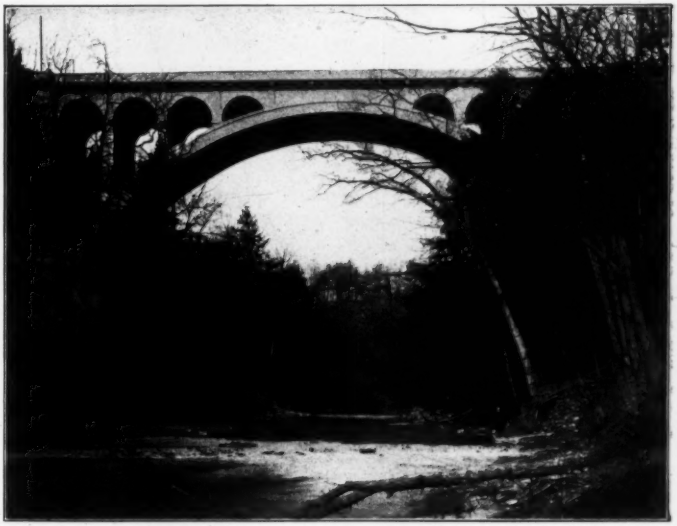


FIG 2.—VIEW OF WALNUT LANE BRIDGE FROM THE CREEK.

main piers with sharp angles in section and overturning stresses, also in the abutment wings, and in the long and slender hand-rails of the balustrades.

The likelihood of concrete becoming detached from large flat surfaces of steel was provided against by the use of steel anchorage at the few points where such action would be possible. These were the projecting ends of the 15-in. floor-beams where they form sidewalk brackets, and the outer side of the 20-in. beams at expansion joints, and the bottom flanges of all beams. The encasing concrete is clamped to the beam webs by ½-in. bent bars passing through holes in the webs, and the flange concrete is tied up with $\frac{5}{16}$-in. wire loops 3 in. apart, connected, and therefore spaced, with side wires welded to them. The concrete of the sidewalk brackets was supplied in addition with ¾-in. square steel loops tying it to the sidewalk slab. The aim throughout the design was to counteract every conceivable tendency of the concrete to open cracks from any cause, and the monumental character of the structure demanded that every precaution be taken to ensure the utmost quality of durability in every part of it.

It is probable that no completed structure of considerable size, simple or elaborate in design, has been regarded by even its author as perfect. Doubtless every progressive designer studies each structure, as it is being built and after it is finished, remembering his reasons for certain features and passing critical judgment upon their merit, and determining how defects may be corrected, or the cost reduced, or the appearance improved. As more is learned from failures than from successes, the most interesting comments on anything may be criticisms of it.

As architectural appearance—beauty of form and grace of proportion—is largely a matter of taste, the author of a work of art is hardly a competent critic of it, and, therefore, suggestions for improvement of the artistic quality of the design of the Walnut Lane Bridge must be left for others. Such modifications of structural details, for economical or other reasons, as have appeared desirable may be of interest to the Profession.

The most persistent enemy of masonry work—stone, brick, or concrete—is water when it finds entrance to it. The provision made in this bridge for drainage has not proved adequate for the complete exclusion of water from the joints, and this—a common failing in such

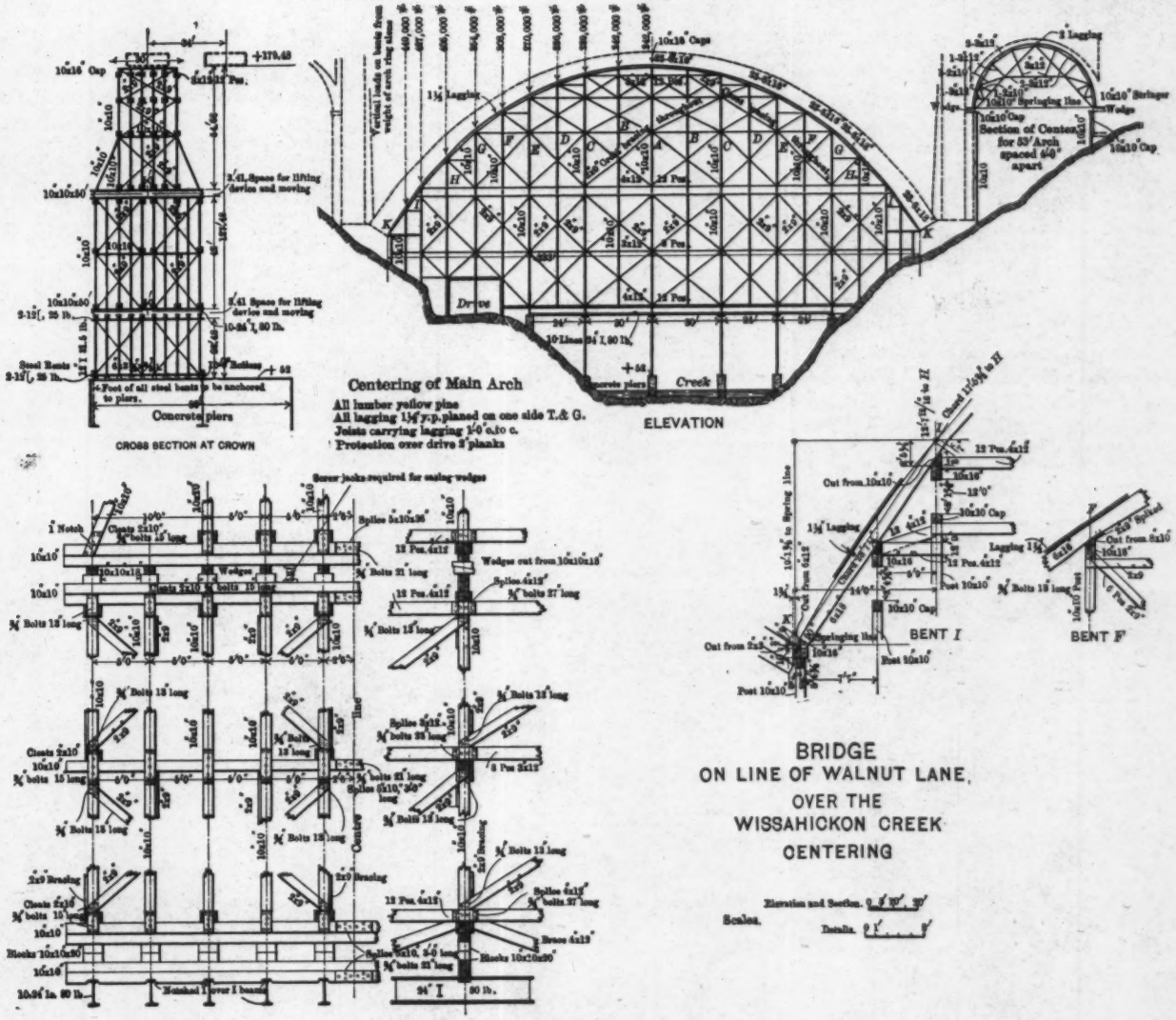
structures—is apparently the only feature of this bridge which can be regarded as a structural defect. The points where it appears, however, are few in number, and the consequences are no more serious than slight blemishes in the appearance. Although the driveway is asphalted and has not a discernible crack in it, some water seems to get through it, and it finds outlet below where it shows on the nearest surface, and produces efflorescence. The joints in sidewalks and curbs are necessarily uncovered, and, consequently, some water finds its way through them, spreads along the floor seat ledges, and appears in stains on the finished faces of some of the abutments and piers. It is intended, in future work of similar character, to construct grooves in the seats and drain them into vertical conductors or down-spouts to be built into the wall joints and below them through the piers or abutments to some available point of discharge at or below the surface of the ground. In other bridges, efforts to prevent the infiltration of water into such joints, by the membrane method of covering the joint with web and elastic coating, have not been entirely successful, but it is believed that thorough drainage of the joints will prevent seepage to the front.

Some economy might have been attained, without lessening the stability or the durability of the work, by omitting the 18-in. concrete sheeting between the ribs of the 53-ft. arches—making four ribs, each 4 ft. wide, instead of two barrels, 18 ft. wide—and by making the spandrel walls 2 ft. thick instead of 2 ft. 6 in. Less steel reinforcement in the piers would have been justified with the high grade of concrete secured by the embedment of the flat stone, which by its complete bonding and great tensile strength resists shrinkage and other rupturing stress.

These are the only modifications of the design which have suggested themselves during the two years of construction of the bridge, which was a period of close observation and test of merit of the design.

The class of concrete adopted and used, together with the character of the surface finish, and also the provision for expansion and contraction, represent the established practice of the City of Philadelphia in its bridge work, and, therefore, were not experimental. The method of constructing the main arch, as prescribed in detail in the specifications, was new, but the results fully justified it as scientific and practical, and it has been used since on similar though lighter work with equally satisfactory effect.

PLATE XXVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



BRIDGE
ON LINE OF WALNUT LANE, OVER THE WISSAHICKON CREEK
CENTERING

Scales. Elevation and Section. 0 4' 8'
Details. 0 1' 2'

The total cost of the bridge, including electrical conduits and lamp standards, bush-hammering, and all extra work, was $267 000, which gives a rate of $7.60 per sq. ft. of floor surface and $0.088 per cu. ft. of space—area of profile by width of bridge.

The prices in the proposal and forming a part of the contract were as follows:

The work covered by the plans—lump sum	$253 550.91
Bush-hammering soffit of main arch ribs—lump sum	1 800.00
Additional bush-hammering—per square yard	0.16
Additional concrete in foundations—per cubic yard	7.50
Additional concrete above foundations—per cubic yard	9.00
Additional steel—per pound	0.04

The bronze lamp standards at the ends of the bridge, and the trimmings and lamps on the iron pipe poles in the refuge bays—contracted for separately—cost $7 982. These poles were made high enough and strong enough to carry diagonal catenaries for the support of overhead trolley wires in the event of street railways crossing the bridge in the future, the purpose being to avoid all use of poles along the curbs. The total cost of bush-hammering the soffits of the main arch, approach arches, and spandrel arches was $2 147.

The total quantity of concrete of all kinds in the bridge is 19 080 cu. yd., of which 2 300 cu. yd. are in foundations and 3 100 cu. yd. in the two ribs of the main arch. The structural steel in the floor amounts to 138 tons, and the steel reinforcing rods in foundations, wings, walls, and floor amount to 54 tons.

The centering of the main arch required 885 cu. yd. of concrete, 334 000 ft. B. M. of lumber, 116 tons of structural steel, and 16 tons of bolts, washers, and nails.

The work of construction extended over two years—from July, 1906, to July, 1908—concrete work having been suspended for three months during each of the two winters. The bridge was dedicated with appropriate public ceremonies and opened to travel on December 16th, 1908, after the grading of the Walnut Lane approaches was completed.

Honorable mention is due to the contractors, Messrs. Reilly and Riddle, for the creditable manner in which they executed the work, and for their resourcefulness and energy in its prosecution.

EXTRACTS FROM SPECIFICATIONS.

The work covered by Item 1 of the proposal includes all the work necessary to make the bridge complete as per plans, and consists of excavation for foundations, grading at Piers 3 and 4, construction of all concrete and reinforced concrete work, with necessary centering and forms, floorwork, paving, iron pipe conduits for electrical wires, drainage appurtenances, manholes, electric light poles, and filling behind abutments as far as the lines shown on the general plan.

The work covered by Item 2 of the proposal may or may not be ordered, and consists of bush-hammering the under surface or soffit of the two ribs of the main arch, with pneumatic bush-hammers, sufficient to obliterate all impressions of the lagging.

In all foundations, stones of derrick size may be embedded, and smaller stones may be embedded with them, but they shall not be closer than 2 in. to any face, and they shall be used only by depositing concrete first and joggling the stones down into it. The embedded stones in the foundations of Piers 3 and 4 shall be of flat shape, and, as far as practicable, inclined so as to be normal to the direction of the resultant thrust on the pier.

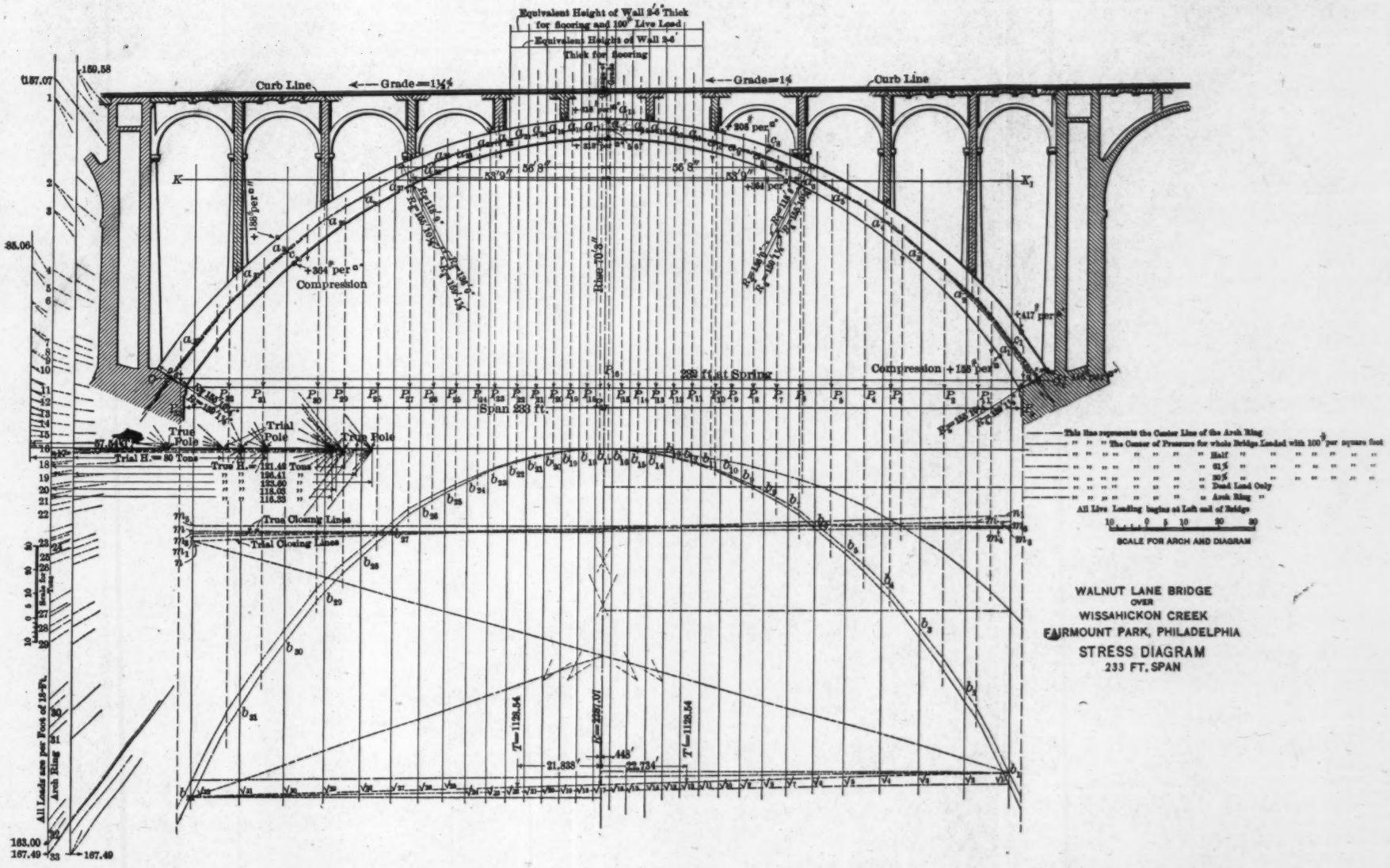
The concrete of the main arch ring shall be composed of 1 part cement, 2 parts coarse sand or gravel, and 5 parts crushed stone, with embedded flat stones. The crushed stone shall be from ¼-in. to ¾-in. in size and thoroughly mixed. The embedded stones shall be not less than one-man size, and of such shape that their thickness will be approximately one-quarter of the average dimension, and shall be of quality and character equal to the best of Conshohocken stone, and be approved as such. They shall be placed in radial planes as close together as practicable, always edgewise to the centering, and embedded by first placing a sufficient depth of concrete and then working the stones into it. At the points of bearing of the spandrel piers and of the spandrel walls a number of these stones shall project above the arch ring to form a footing and bond. The concrete of the 20-ft. spandrel arches and the 53-ft. approach arches shall be of the same composition, with similar stones, of the same character, and of suitably proportioned size, similarly embedded.

All vertical walls of neat work shall have embedded in the concrete approved hard stones, not less than one-man size, so that they approach not closer than 2 in. to any surface, and so that every course or layer will have a number of the stones projecting up from it to dowel or bond with the succeeding course.

Piers 3 and 4 must be complete to and including the skewbacks at least one month before the concrete of the main arch is deposited.

Each main arch ring shall be constructed in transverse sections in pairs—opposite sides simultaneously—to ensure symmetrical loading on the centering. These sections shall be of such length that each pair

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXV, No. 1128.
WEBSTER AND QUIMBY ON
WALNUT LANE BRIDGE.



WALNUT LANE BRIDGE
OVER
WISSAHICKON CREEK
FAIRMOUNT PARK, PHILADELPHIA
STRESS DIAGRAM
233 FT. SPAN

can be constructed in daylight of one day. A detailed plan showing the sections and the required order of construction will be furnished by the City. The sections shall be separated by spaces of about 2 ft. 6 in., which will constitute the keys of the arch. The keys shall be constructed not earlier than two weeks after the last pair of sections has been placed. The lower three key spaces on each side shall have on the mid-line three concrete piers constructed at the same time as the sections, to maintain the separation of the sections and prevent possible sliding on the lagging. These piers will be surrounded and built in by the placing of the concrete of the key. The joints between keys and sections shall be locked by tongue and groove 1 in. deep.

The design of the centering to be used shall be submitted for approval before proceeding with its construction. The falsework for the arches shall rest on rock or on masonry piers built on the rock. The design shall be such as to avoid, as far as practicable, bearing across the fiber of the wood, confining the pressure as far as possible to the end fiber, in order to reduce to a minimum the yielding of the structure as it receives the load. Means shall be provided for lowering the supports evenly and gradually under absolute control.

Expansion joints in the walls and all joints in the balustrade shall be locked by tongue and groove 2 in. deep, and have $\frac{1}{8}$ in. thickness of asbestos felt separating the surfaces. The balustrade joints occurring at floor joints shall have $\frac{1}{4}$ in. of asbestos felt and have the locking tongue only slightly beveled to permit free movement.

At the expansion joints in the floor, where the movement involves sliding of the floor slabs on the walls, the bearing of the sliding portion shall consist of a strip of $\frac{1}{16}$ in. thick zinc, of the width of the bearing, flanged up 1 in. at the back edge.

At the expansion joints in the sidewalks the electrical conduit pipes shall be jointed by wrapping each with 4 by $\frac{1}{32}$-in. zinc to permit movement.

The connection between the 15-in. beams and the 20-in. beams of the floor must be made with tapered, turned bolts. The connecting angle is to be riveted to the web of the 15-in. beam with four $\frac{7}{8}$-in. rivets, and the holes in the flange of the 20-in. beam are to be drilled in position and reamed tapered to fit the bolts, which shall be not less than $\frac{7}{8}$ in. in diameter at the point of shear.

There shall be embedded in the concrete around the bottom flange of each floor-beam $\frac{3}{16}$-in. wire loops 3 in. apart, connected on each side of the beam above the flange by two longitudinal wires $\frac{1}{8}$ in. in diameter, and 3 in. apart, to hold the loops in position. There shall be no longitudinal connection below the flange. The longitudinal wires shall be secured to the loops either by welding, as in the "Clinton Wire Cloth," or by tightly wrapping, as in the "Wight Lock Woven Steel Fabric."

The vertical drainage pipes from the grate top inlets shall be 10 in. in diameter, 40 lb. per foot, asphaltum-dipped, wrought-iron or steel pipe. The drainage pipes through the foundations of Piers 3 and 4 and underground shall be 12-in. and 16-in. cast-iron pipe. The latter shall be laid with 3 ft. of earth over it, and shall extend to the rocks at the water's edge of the creek.

The electric light poles shall be complete, except lamps and wires, which are not included in this contract. The poles and fittings shall be ornamental, and the design shall be submitted and be subject to approval before erection. They shall be of such height that the lamps will be 21 ft. above the sidewalks. They shall be anchored and built into the concrete work as it proceeds. The poles over Piers 3 and 4 shall be of sufficient strength to serve as trolley poles.

The steel edging for the granolithic curb shall be furnished in lengths to match the joints in the sidewalks.

The whole surface of the floor concrete between the curbs, and also the buried portion of the face of the curbs, shall be thoroughly coated with No. 4 coal-tar distillate to a least thickness of $\frac{1}{4}$ in.

Bush-hammering of surfaces, if required to be done, shall be with pneumatic bush-hammers, and shall be sufficient to remove the surface film, exposing the grit, and to remove the impressions of joints of the lagging.

The contractor shall furnish, at points to be designated, in the arch centering, wooden tell-tale pendants with corresponding stakes in the ground to facilitate accurate observation of deflection as the work proceeds.

Where anchorage in the concrete is required to maintain the forms in true position, it shall be designed so that any projecting metal work can be removed to a depth of 3 in. from the face without damaging the face, and the cavities thus created shall be filled with granolithic mixture.

The grit in all granolithic face work, except the balustrade, shall be of dark hard stone from an approved quarry, and that in the balustrade shall be of crushed granite.

The cinder filling in the middle section of floor shall be selected material, and shall be thoroughly compacted by wetting and tamping.

To economize in centering and forms, the bridge superstructure may be constructed in halves—one side throughout the whole length at a time—but the two abutments and the foundations of all piers must each be monolithic.

The following additional notes by the authors have been prepared in consequence of several questions asked at the meeting at which the paper was presented:

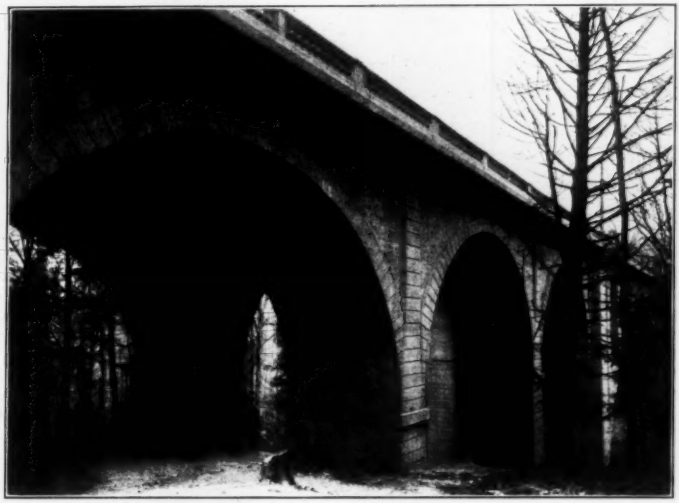


FIG. 1.—WEST APPROACH SPANS, WALNUT LANE BRIDGE.

FIG. 2.—EAST APPROACH SPANS, WALNUT LANE BRIDGE.

The fixed or hingeless design for the main arch ribs was adopted because of its economy and its graceful appearance. The nominal advantage of hinges is the elimination of temperature stresses. The disadvantages are the awkward appearance resulting from the necessary thickening of the ring at the quarters, and the extra cost of the hinge construction. Pivots confine the line of pressure to the mid-line of the ring at the spring and at the crown, which causes a much wider range of it at the quarters, necessitating a much greater thickness there than less eccentric compression would require. The square-ended ring bearing at the spring and at the crown facilitates the adjustment of the center of pressure throughout the whole length of the arch, where it is demanded by variations in loading. Hinges have their field of usefulness in flat arches where changes in length of arc caused by variations in temperature cause more vertical movement at the crown than they do in arches with considerable rise. The proportion of rise to span in this bridge is so great that the temperature stresses are very light and practically negligible—about 75 lb. per sq. in. on extreme fibers at the points of maximum effect. If hinges had been introduced, the thickness of the arch ring at the quarters would have been increased about 2 ft. in order to avoid increasing the unit stress; and, at the spring, it could not have been reduced more than 3 ft., and the slight saving in concrete would not have paid for the hinges. Besides, the graceful sweep of its present lines could not have been obtained.

In a dry voussoir arch, such as this, there cannot, of course, be any temperature stress, strictly speaking, because the ring is merely a pile of blocks most accurately fitted together, but not adhering, and, consequently, not competent to resist any bending force, and where there is no resistance there is no stress. Therefore, the only effect of temperature change is to throw the load stress toward and upon one or the other of the edges by deforming the arch, as described in the paper. Examinations of the whole surface of each main arch rib have been made in extremes of atmospheric temperature, but not one of the forty joints in each of the ribs has shown any opening, which is good evidence that the center of pressure has never been thrown to either edge of the middle third—the eccentricity of compression has not been as much as one-sixth the thickness of the ring at any point.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1129

HAS EXPERIENCE DEMONSTRATED THAT THE OILING OF ROADS IS THE MOST SATISFACTORY OR ECONOMICAL METHOD OF PREVENTING DUST AND PRESERVING THE ROAD SURFACE?

An Informal Discussion at the Annual Convention, July 8th, 1909.

By Messrs. S. Whinery, and G. S. Davison.

Mr. Whinery.

S. Whinery, M. Am. Soc. C. E.—We have now had sufficient experience with the use of oils and tars for dust prevention and road preservation to make it worth while to take stock of the facts available, to consider their import, and to form some tentative conclusions as to the efficiency and economy of the process. Only one other process or method of accomplishing the same results is available for comparison, and that is watering or sprinkling. It is true that modifications of these two leading methods have been used more or less. Among these is the use of a solution of some deliquescent salt in water, and an emulsion of oil in water; but while these have each been reported as having given satisfactory results in a number of cases, it can hardly be claimed that their use is general or extensive, and the verdict of experience is mostly against them. In any event, it may be said that they are only modifications of the two primary methods, and there is good reason to believe that none of them will be found more satisfactory or, on the whole, more economical than the use of oil or water alone.

The discussion, therefore, is narrowed down to a question of oil or water. It may also properly be confined to the treatment of ordinary broken stone or macadam roads, in the eastern section of the

Mr. Whinery.

United States, which carry a sufficient quantity of travel to make them very dusty in dry weather, and to cause them to wear and disintegrate rapidly, unless they are properly cared for.

It is known from long experience that the dust may be kept down by sprinkling with water, and that if the body of the macadam be kept properly moist, abrasion from travel will be largely, and that kind of disintegration called "raveling" almost wholly, prevented. This is particularly true when the bulk of the tonnage passing over the roads consists of motor-driven vehicles, as the action of their rubber-tired wheels is quite different from that of steel-tired wheels. In recent years the opinion has become quite general that equally good, or better, results can be accomplished more economically by applying oil or tar to the surfaces of roads. The question under discussion, therefore, may be resolved into this: Can dust be avoided, and can the wear and raveling of macadam roads be prevented more satisfactorily and more economically by the use of oil than by the use of water?

To consider this question fairly, it must be assumed that each method shall be intelligently and skilfully used, or, in other words, that each shall be given a fair chance in the contest between them. It would be obviously unfair and unprofitable to judge the merits of either from the results of average practice, where that average is largely affected by ignorance or carelessness. In most cities and towns street or road sprinkling is little short of a delusion and a snare for the municipal funds. Little attempt has been made by municipal engineers to study, systemize, or regulate it in a scientific manner. Statistics of its unit cost, and of the quantity of water required, have been difficult to get, because sufficient attention has never been given to the work to insure the collection of accurate data. In the case of road oiling, the absence of knowledge and skill, and not infrequently of ordinary good sense, has led to a multitude of failures and to the waste of large sums of money. But in this work, thanks largely to the United States Office of Public Roads, and the different State road departments, there are quite full and reliable cost data, though not very many of these data relate to what is now considered the best practice.

In favor of street or road watering, it may be said that, if properly and intelligently applied, it will effectually suppress dust, and will prevent the disintegration of the road material by raveling. It is, in fact, almost unanimously agreed among good road engineers that raveling is caused by the absence of a proper amount of moisture in the macadam material. It is doubtless true that water does not prevent surface abrasion as effectually as oil, but with the large ratio of rubber tires now found on the roads, abrasion in the true sense of the word is not a very important element of road deterioration.

Mr. Whinery. The most serious objection to watering is that when improperly done, that is when the street is flooded with a surplus of water, it is likely to be sloppy and disagreeable for pedestrians, and the wheels and bodies of vehicles may be spattered with mud, which, however, is readily removed by washing with water alone. One advantage of some importance is that the material for watering streets is always at hand, and the plant required is simple and usually available at any time, so that the roads can be sprinkled at any time during the year when it may become necessary. It is also an advantage that travel need not be interrupted or interfered with, while the water is being applied, or afterward, and that no special preparation of the street surface is necessary.

Oiling is as effective as watering for suppressing dust, if the whole road surface is oiled. But usually, because of the cost, only a strip in the middle of the road is treated, leaving untreated strips along each side, which may be troublesome sources of dust in dry weather if the travel is heavy. Oiling cannot be done properly unless the road be perfectly dry and the weather favorable, and for this reason it is thought expedient, if not absolutely necessary, to postpone the treatment until quite late in the season. In the meantime people must endure the dust or resort to watering for from one to two months in the spring. The surface must be carefully cleaned and prepared for the reception of the oil, and, after it is applied, the road, or the oiled part of it, must be closed to travel for from two days to a week. Even after it is opened to travel, the surplus oil still on the surface makes it very disagreeable for pedestrians for several more days, and the free oil or oil-saturated dirt is thrown by the wheels over the vehicles and the clothes of the occupants, and these greasy stains are very troublesome to remove. In the autumn and winter season following, these oiled surfaces produce an abundance of oily slime and mud, creating a condition for which the word "nasty" is an inadequate term, and since the remaining oil is sufficient to make this mud more or less cohesive and water-resisting, it is not carried away by the rains, and continues through most of the winter.

The extent to which oil preserves the road surface from wear and disintegration is not yet definitely determined. As long as the oiled surface remains unbroken there is practically little loss by abrasion. That oiling does not prevent, but rather promotes, a kind of disintegration analogous to raveling, on heavily traveled roads, is evident to those who have observed the effects carefully. This result might be expected and predicted on theoretical grounds. Raveling is caused by the drying out of the body of the macadam. The oiled surface, compressed by travel, forms an impervious coating over the road through which the rain cannot penetrate. The result is that the macadam gradually loses its moisture and consequently its cohesion, and the dense surface coating is broken up and comes off in patches,

carrying with it a stratum of the distintegrated macadam surface. These patches enlarge rapidly and others appear, so that roads carrying a heavy travel during a dry season are likely to become in bad condition before the winter begins. Mr. Whinery.

This statement may seem overdrawn to many, but it correctly represents conditions actually observed during the season of 1908 on first-class macadam roads well treated with the best quality of oil, in an eastern city. The speaker has no hesitation in saying that at the end of the season the physical condition of the oil-treated sections of these roads was not better than the condition of adjacent sections of the same roads, which were sprinkled more or less ineffectually.

Even if it were true that oiling is, on the whole, rather more efficient than watering, the comparative cost of the two processes must be given proper consideration before arriving at a final decision.

Reliable and accurate data relating to the cost of street sprinkling are rather meager. Even where prices and areas are given with some care, so many other factors and conditions are usually omitted that it is impossible to deduce satisfactory unit costs. A careful study of the data available, supplemented by careful estimates of what the cost should be, leads to the conclusion that on macadam streets carrying a fairly heavy travel, the work as now usually done does not cost more than 2 cents per sq. yd. per season. In a number of cities where the work has been done with more than the usual care, the cost has been below that figure, which includes the cost of the water used, charged at $75 per million gallons. Street sprinkling, however, is very seldom properly done. The sprinkling is too infrequent, and the quantity of water applied at each sprinkling is too great. Where the surfaces are ordinarily sprinkled twice daily, they ought to be gone over three times daily in order to secure the best results; but, on the other hand, with proper care to avoid waste, the quantity of water used might be materially reduced. One may often see twice the quantity of water actually necessary being discharged over the street, and the surplus half running into and flowing away along the gutters. It may be safely stated that, with the more frequent sprinklings needed and with the work done with the same care and skill necessary for successful road oiling, a road of the character just considered can be watered so as wholly to avoid dust, and to preserve the road itself at a cost not exceeding 3, or, at most, 3½ cents per sq. yd. per season. It should not exceed the lower figure under efficient management.

In a large number of cases the cost of oiling roads has been accurately recorded, and the unit costs have varied as widely as the character and quality of the work. In what is now regarded as the best practice, using a high-grade asphaltic residual pitch, with the necessary preliminary preparation of the road surface and the proper

Mr. Whinery. application of the oil, a cost of from 5½ to 6 cents per sq. yd. may be regarded as reasonable for a single effective application. Such an application made between the middle of June and the first of July should be effective, on the class of roads being considered, until the end of the usual sprinkling season, but its effect will almost wholly have disappeared before the next spring, when the treatment must be repeated *de novo*.

During the early part of the season, say from the first of May, when sprinkling is usually started, to the middle of June, which is about as early as oiling is practicable, sprinkling must be resorted to or the dust endured. Disregarding this, however, and calling the cost of efficient sprinkling 3½ cents per sq. yd. per season, and the cost of oiling 5½ cents per sq. yd. per season, it appears that the oiling costs 57% more than the watering. The speaker does not believe that experience has yet demonstrated, or will in the future demonstrate, that, everything considered, oiling is 50% more efficient or satisfactory than sprinkling, when each is equally well done.

If the statements and figures here presented are reasonable and sound, the question seems to have been answered, and this other question is suggested: Is road oiling anything more than a popular fad of the day, unworthy of the further serious attention of engineers?

Engineers are all looking for the advent of some new kind of road, which, while not too expensive for use, at least on the more important roads, shall be more durable than macadam and shall not generate dust. The solution of the problem does not seem far off. Such roads may be truly economical even though their first cost seems high; they will certainly be cheaper in the end than oiled macadam roads. The payment of 6 cents per sq. yd. per annum for oiling represents an investment, at 6%, of $1.00, and it is reasonably certain that less than $1.00, added to the first cost of the plain macadam, would provide a road which would not need oiling. The wise course, then, is to give more attention to constructing new roads which will be practically dustless, and to be content with the cheaper and quite as effective method of watering to combat the dust on the old roads.

Mr. Davison. G. S. Davison, M. Am. Soc. C. E.—The speaker would like to ask Mr. Whinery what part of the cost of oiling the roads in question, was freight on the oil. In certain cases this freight item might make the use of oil prohibitive.

The greater part of the oil used for this purpose is from Southern refineries located near the seacoast, and is sent to Boston, New York, Philadelphia, and such points, with very low freight charges. Such oil will come within reasonable figures, when the cost of reshipment is low. Therefore, unless the oil for roads in the interior of the United States can be obtained from refineries close at hand, the oiling of roads must be confined to points near the seacoast.

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TRANSACTIONS

Paper No. 1130

CONCRETE PILES.*

By Howard J. Cole, M. Am. Soc. C. E.

With Discussion by Messrs. Morton L. Tower, L. J. Mensch, Charles H. Higgins, Maxwell M. Upson, Thomas C. Desmond, R. D. Coombs, Homer A. Reid, Hunley Abbott, and Howard J. Cole.

The rapid decrease in the available timber supply—and the consequent increase in the price of all kinds of wood during the past thirty years—has served as a stimulus for the invention of methods for the conservation of this valuable building material. Investigations have proceeded along two distinct lines; first, to find a means whereby the serviceable life of the wood could be prolonged; second, to find a material which could be used as a substitute for wood. The first was accomplished by the discovery of the creosote treatment, which has more than doubled the actual term of service of wood in those industries in which it has been used. The second problem has been solved to a large extent by the development of the uses of concrete, both plain and reinforced, which is now replacing not only wood, but also steel, masonry, and other materials which have been standards in construction work for years.

The construction of concrete piles is one among the many uses to which concrete has been applied successfully as a substitute for wood. The first patents, in the United States, were taken out within the last decade. In that short period concrete piles have grown rapidly in favor, and are now replacing timber piles for the support of heavy

* Presented at the meeting of September 1st, 1909.

structures in wet or marshy situations. One concrete pile company states that during the past year its business has increased several hundred per cent., as compared with the previous year, although the period has been one of unusual financial depression.

It is a well-known fact that timber piles will last practically forever when constantly submerged in water, and there are on record many authentic instances where they have been found intact after a service of several hundred years. Probably the most conspicuous example that has been brought to the attention of the world in recent years is the foundation of the Campanile of St. Mark's, in Venice, built in 900 A. D., the superstructure of which fell in 1902 on account of weakness of the masonry due to other causes, and not from any fault in the pile foundation. The piles were found to be in such good preservation that they are being used to support the reconstructed tower. These piles were installed under favorable conditions, and were constantly submerged, but where timber piles are subjected alternately to the action of air and of water they soon decay.

The lowering of the ground-water level around a structure supported on timber piles will eventually cause damage to it and possibly its destruction. Many instances of this kind have been caused by the construction of the numerous tunnels and subways in New York City, as the level of the ground-water in their neighborhood has been reduced so much that the stability of pile foundations near them has been menaced. One very prominent building, constructed on timber piles only a few years ago, in a location which naturally required a foundation of that type, is endangered from this cause and, in the near future, steps will have to be taken to strengthen the foundation.

Where hard driving has been necessary in order to place timber piles in position, engineers have entertained a doubt as to the final condition of the pile and its real ability to support the load imposed on it, owing to the fact that many piles driven under these conditions and removed later have been found to be damaged seriously by being either telescoped or buckled. Recently, during the construction of a pneumatic foundation in New York City, a line of caissons was sunk along the side of the lot directly against the adjacent building, which rested on a timber pile foundation. A number of the piles had been driven so that they projected over into the other lot, and these were removed as the caissons were sunk. Some of these piles were buckled

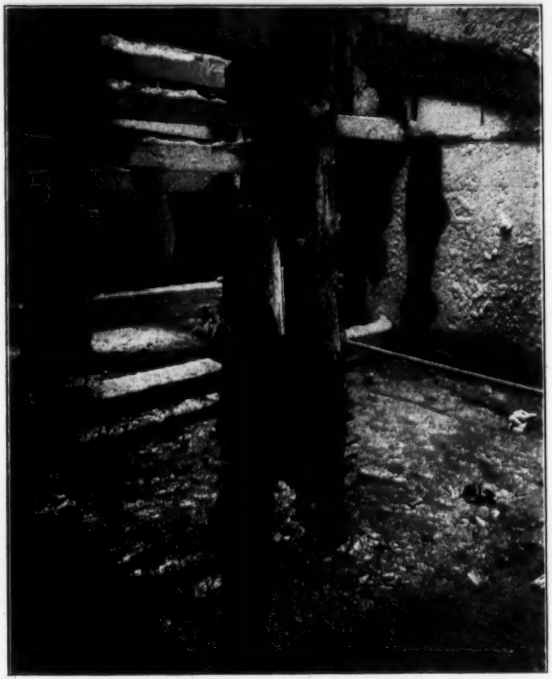


FIG. 1.—TIMBER PILES DAMAGED IN DRIVING: EXPOSED BY EXCAVATION.

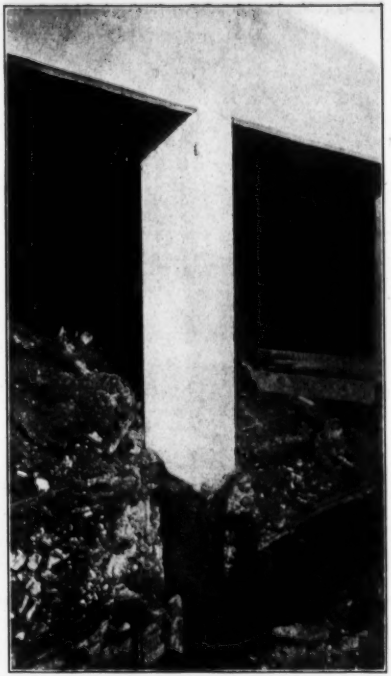


FIG. 2.—SUPERSTRUCTURE FOR OIL TANK COLUMNS SUPPORTED ON TYPE A PILES OF THE MADE-IN-PLACE CLASS.

and crimped into accordion-like folds, and had no sustaining power whatever; how many more are in a similar condition is a matter of conjecture. Timber piles are injured more frequently in driving than is generally supposed; but comparatively few are ever removed, and the integrity of the foundation is taken as a matter of fact. In the construction of caisson foundations for sky-scrapers in New York City, it has been the custom in many instances to drive timber piles to form a temporary support for the derrick traveler. These piles remain in place until all the material has been excavated, and are then removed; and, although they have been selected carefully and driven by expert workmen, a large percentage is found to be badly damaged.

In the construction of piers, docks, and similar structures, if timber piles are installed in waters frequented by marine borers—the teredo or the limnoria—their life is very short; in badly infested waters an untreated pile may not last eight months, while a creosoted pile may last for years, though its period of service is somewhat doubtful, as piles similarly treated and placed do not last the same length of time.

The concrete pile has none of the disadvantages of the timber pile, as it cannot be affected by the reduction of the ground-water level, or by the attacks of marine borers, nor is it injured by hard driving. It can be driven into ground impenetrable to the timber pile, can be made of the dimensions and form desired, and can be proportioned to the load that it is to support; at the same time, every pile can be made to comply fully with the specifications. Those who have inspected timber piles realize how difficult it is to obtain pieces which will fill the demands of the specifications in all particulars, as to size of butt, size of point, straightness, etc., especially when lengths of more than 50 ft. are required. The concrete pile has a greater carrying capacity than the timber pile, individual piles having been tested with pig-iron loadings of 45 tons and more without settlement, and piers of five piles have supported satisfactorily a test load of 300 tons, or an average of 60 tons per pile. In practice, however, it is not desirable to load them to more than 25 tons per pile, unless reinforced to provide for the excess loading. In good practice, timber piles of a comparable size would not be loaded to more than one-half of this amount, therefore the carrying capacity of the concrete pile is double that of the timber pile.

The method adopted in the United States to demonstrate the carrying capacity of a concrete pile has been to make an actual loading test of some of the piles by placing a platform on a pile, loading it with pig iron to double the guaranteed load, and, by a series of accurate levels, keeping watch for any settlement that may occur.

Some attempts have been made to calculate the supporting power of concrete piles by the formulas generally used with timber piles, but it is quite evident that none of these formulas will apply in cases where the concrete pile is jetted into place. When it is driven into position, however, a formula like "Ritter's," which takes into account the weight of the pile, can be used with good results. The "Ritter" formula is:

$$W = \left(\frac{h}{e}\right) \frac{Q^2}{Q+q} + Q + q,$$

in which W = The resistance to further penetration,
Q = The weight of the hammer, in pounds,
q = The weight of the pile, in pounds,
h = The height of the fall, in inches,
e = The penetration, in inches, by the last blow.

The concrete pile costs more per linear foot than the timber pile, but, in many installations, some items of construction, necessary with the timber pile, are eliminated entirely or reduced materially by the use of the concrete pile, so that the final result shows a considerable saving in time and money with the latter. For instance, where timber piles are used and the ground-water is several feet below the surface, it is customary to excavate to the water level and support the sides of the excavation with sheathing before driving the piles; then, after the piles are in place, they are cut off at the proper elevation, the pier or wall is built to the required height, and the excavation is back-filled. This proceeding is simplified greatly with the concrete pile foundation, where the pile is driven from the surface of the ground, without any preliminary excavation, and only requires a capping extending below the frost line to prepare it for the superstructure. Under these conditions, the concrete pile foundation saves on the items of excavation and depth of pier while it eliminates entirely the items of sheathing, pumping, and back-filling, and at the same time accomplishes the result in a much shorter period of time.

The ideal location for a concrete pile is in filled areas where the water level is at considerable distance below the surface and the ground is of such a nature that it is necessary to carry the footings to a firm underlying stratum; here, the concrete pile can compete with the usual concrete or masonry pier, as, under these conditions, it will save most of the items mentioned in the example just given. In the construction of foundations for a large warehouse at Pittsburg, Pa., on the site of an old fill, from 20 to 30 ft. deep, where it was necessary to carry the foundations to a firm stratum, bids were received on two methods of construction: one being to carry the loads to the proper depth by concrete piers in open excavated pits, and the other to use concrete piles. The lowest time limit set by the open-pier method was 18 months; the work was actually done by the concrete-pile method in 76 days, thereby effecting a saving of more than 15 months in time alone, as well as a considerable amount of money.

Comparisons of the relative cost of concrete and timber pile foundations in fair-sized installations are not common, but an instance is cited in detail of a foundation at Annapolis, Md., which shows a saving in money of more than 50% on the cost of a timber pile foundation:

2 193 timber piles at $9.50	$20 833.50	
4 542 cu. yd. excavation at $0.40	1 816.80	
3 250 cu. yd. concrete at $8.00	26 000.00	
5 222 lb. I-beams at $0.04	208.88	
Shoring and pumping	4 000.00	
		$52 859.18
855 concrete piles at $20.00	$17 100.00	
1 038 cu. yd. excavation at $0.40	415.20	
986 cu. yd. concrete at $8.00	7 888.00	
		25 403.20
A saving of		$27 455.98

The saving in material was:

1 338 piles,
3 504 cu. yd. excavation,
2 264 cu. yd. concrete,
5 222 lb. I-beams, and
all shoring and pumping.

Although the concrete pile is cheaper under favorable conditions, the timber pile will be used for many years, especially in places where the ground-water level is near the surface and the building to be supported is so light and cheap as not to warrant an expensive foundation.

There are many kinds of concrete piles on the market. Most of these are patented and are the varieties in general use; they can be divided into two general classes, according to their method of construction: First, those made in place, where the pile hole is made by a mandrel and the concrete is dropped directly into the position it is to occupy as a concrete pile and sets in place; and second, those moulded or cast and allowed to harden thoroughly before being placed in position. For piles of this class the pile hole is not prepared in advance, but the pile is driven or jetted into place in a manner similar to that used with timber piles.

These piles all differ from each other in shape, being either straight or tapering, and in cross-section being square, polygonal or circular. These variations in form have led to considerable discussion as to the merits of different combinations, but it is the intention in this paper to mention in an impartial manner only the relative bearing powers of straight and tapering piles, without going into the comparative merits of the others, leaving that element to be brought out in the discussion, with the details of construction, cost figures, etc.

In the first class to be described there are two representative patented types. Both of these were first patented and used in the United States, and are distinctly American; consequently, installations of this class have been more numerous than of the second or moulded class. One of these, which for purposes of identification will be called Pile "A," is formed by driving to refusal a hollow steel pipe or form, of the proper length, $\frac{3}{4}$ in. in thickness of shell, 16 in. outside diameter, and of the same diameter throughout its entire length. This hollow form is driven by an extra heavily built pile-driver, which, in addition to the usual equipment, has a strong pulling device attached to the leads for the purpose of withdrawing the form from the ground after driving. The lower end of the form has a special jaw or shoe which is closed during driving, in order to prevent the entrance of the surrounding materials into the form; in certain soils the "Alligator Jaw" point is used. This jaw is attached securely to the form by

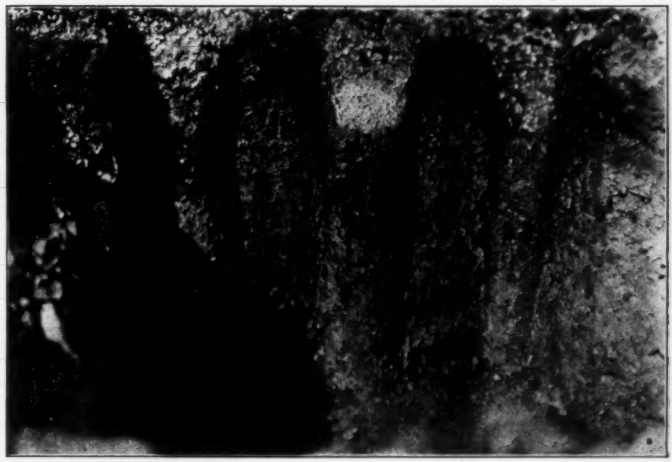


FIG. 1.—TYPE A, MADE-IN-PLACE CLASS: PILES EXPOSED FOR INSPECTION.

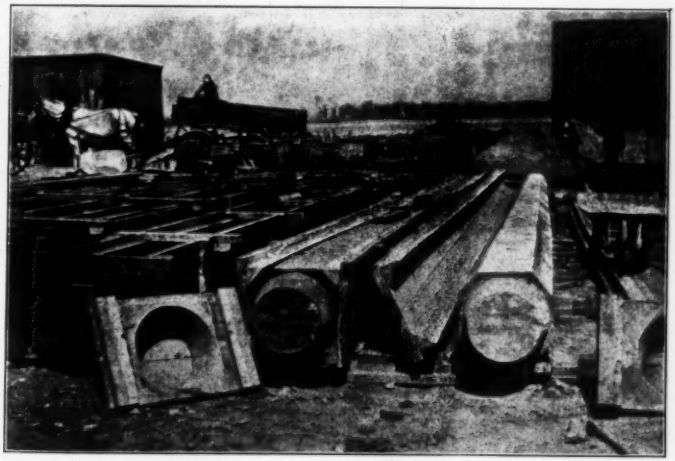


FIG. 2.—MOULDED PILES, SIXTH STREET VIADUCT, KANSAS CITY.

cable hinges, and is used over and over again. It is composed of two symmetrical wedge-shaped parts which are kept closed during driving by the pressure of the soil. Generally, a detachable cast-iron point is used, but this is more expensive than the "Jaw," as it is left permanently in place, and one must be supplied for each pile. The upper end of the form is reinforced with a strong band of steel shrunk in place and secured firmly to the form with countersunk rivets; this steel band also serves as a shoulder for the clamps which are attached to the pulling device. A steel driving head, sliding in the leads, with a circular projection on its lower face to fit into the top of the form, is lowered on the form, and not only serves to hold the top of the form firmly in place during driving, but also receives the impact of the hammer, from which it is protected by a rope mat.

When the form has been driven to refusal, the driving head is hooked to the hammer by a wire attachment, and both are hauled to the top of the leads and chocked there during concreting operations. A charge of concrete—about 5 cu. ft.—is dropped into the form, after which the entire form is raised about a foot by the pulling device; then a heavy iron rammer is dropped on the mass and by its impact opens the jaw and forces the concrete out into the space made by the end of the form. This procedure is repeated until the form has been entirely withdrawn from the ground and the space it occupied is completely filled with concrete, which hardens in place and becomes a concrete pile.

This method of filling is varied to meet varying conditions; occasionally, after the jaw has been opened, a sufficient quantity of concrete to make a complete pile is placed in the form and the form is steadily withdrawn; this plan, under certain sub-surface conditions, gives a better shaped pile, because, under the first method, the jaws have a tendency to close partly on account of the excess of pressure at each stop of the form, thus making a pile of irregular section; this is avoided by giving a regular and steady movement to the form. The concrete is made quite wet, of a 1:2:4 mixture, using ¾-in. stone, and, when placed in the form in mass, is heavier than the surrounding soil. Thus it is more than able to balance the pressure, and the soil has no tendency to cut off any section of the pile, as the excess pressure serves to force the concrete into the soil.

In very hard driving, heavy forms, even with a ¾-in. shell, have

been buckled at the top, and one can judge from this what the condition of a timber pile would have been under similar conditions. In one case the form was driven into the lower stratum so firmly that the pulling device could not withdraw it, so it was filled with concrete and left permanently in place.

It is claimed for piles of this type that the fresh concrete, which comes into direct contact with the surrounding material, enters into all its irregularities and unites with it, thus forming a rough exterior surface, which gives the pile a large amount of frictional resistance; and, further, as practically all the resistance to driving is confined to the point, the greater part of the compression takes place where the pile needs it most in order to transfer its load as a column.

In locations where the strata penetrated by the pile are in a fluid or semi-fluid condition, and have not enough consistency to serve as a mould to retain the concrete in shape, piles of this type could not be used without material modification. In one case of this nature, after the driving form had passed through the soft soil and had been driven to refusal into the firm underlying stratum, the initial charge of concrete was inserted and the point was forced open; then an auxiliary form of thin No. 20 metal, of slightly smaller diameter than the driving form, was dropped into place, and the required reinforcing was adjusted. Then the auxiliary form was completely filled with concrete, and the driving form was withdrawn, leaving the thin shell in position to act as a mould for the concrete. The foundations for a public utility building on Staten Island, N. Y., were constructed in this manner, the resulting pile being one which had its point embedded in the bearing stratum 8 ft. and was 16 in. in diameter in this portion, while the column part of the pile was 12 in. in diameter, and was enclosed in a shell of No. 20 metal. These piles stood successfully the test loading.

The length of piles of the ordinary type driven by this system is between 20 and 25 ft., but piles 48 ft. long have been used. The largest installation of this type is in the foundation of the Pittsburg Terminal Warehouse, Pittsburg, Pa., where about 6 000 piles, from 30 to 45 ft. long, were placed.

The other leading type of this made-in-place class, Pile "B", differs considerably from the one just described. It is tapering in shape, and each pile has a steel shell, to prevent the earthy material from

mixing with the concrete, this shell being left permanently in position. A heavy pile-driver, usually on a turn-table frame for facility of operation, is used to drive a tapering collapsible form or core into the ground to make the pile hole; this core weighs about 3 tons, and is built in three longitudinal sections with provision for horizontal movement that permits the sections to be expanded or collapsed. Before beginning operations the core is suspended in the leads, with the point about 8 ft. above the ground to allow clearance for the sections of casing to be slipped on. The casing is made from sheets of No. 20 metal, 8 ft. long, which are formed into the required shape, the edges being bent to connect together and retain the shape of a truncated cone. These are of various sizes, to fit the tapering core, and are slipped on in regular order with a considerable overlap to each section, enough sections being used to cover the core completely to the length required. Occasionally, the joint between the sections is covered with an extra section to make a tighter joint. When this is done, a pressed-steel boot or thimble is forced on the point of the core overlapping the last section of casing. The core is now ready for driving. It is slung back into the leads, lowered carefully to the proper location on the ground, and is then driven to refusal by a steam hammer weighing about 3 tons. When the core has been driven to the desired depth, it is collapsed and withdrawn, leaving the steel casing in the ground as a lining for the pile hole; this is immediately filled with 1:2:4 concrete, using $\frac{3}{4}$-in. stone or gravel, mixed rather wet, which sets and forms the concrete pile. The concrete cannot be placed until the core has been withdrawn. In practice, it has been found that these protecting shells, being sectional, are not tight, but permit the entrance of water and quicksand which has to be siphoned out before the concrete is placed. When installed in locations where the shells are subjected to great hydrostatic pressure, the casing is sometimes collapsed and must be re-driven. In very hard driving in boulder formation, the core has been injured so severely as to be put out of commission and has had to be replaced by another.

It is claimed for this pile that its strong tapering shape, 20 in. at the butt and 8 in. at the point, distributes the load throughout the entire length of the pile and compresses the soil to such an extent that the full bearing of the upper layers is fully utilized, and also, that the metal casing prevents the surrounding soil from mixing

with and weakening the concrete. The Hospital Buildings of the United States Immigration Commission, at Ellis Island, New York Harbor, were installed by this method.

These two varieties of the made-in-place class have been more popular than any others in the United States in building foundations. They are reinforced occasionally with rods which are put in position as the concrete is deposited, or single rods may be churned into place through the wet concrete. The rods are left with their ends projecting above the concrete 1 ft. or more, in order to assist in bonding the capping to the piles. This made-in-place class is not adapted for use in docks, piers, wharfs, etc., without very material modifications of the typical pile described.

The installation of these piles has not always been a success; there have been failures, due to various causes, from which much has been learned. In one case all the subsoil conditions were favorable, the driving was satisfactory, and everything indicated a very successful piece of work; but when the physical test was applied the piles failed to support the load. On investigation it was found that the fault lay with the cement. One of the standard brands had been used, but it had been accepted without test, and proved worthless. In other cases difficult sub-surface conditions were met, each of which required some change in operations; but, at the present time, an unsuccessful installation is rare when all the conditions of the problem are carefully studied and investigated.

No single type of concrete pile will fit all locations without some change to suit the site, and, before determining what particular pile shall be used, a very careful survey of the subterranean strata should be made by taking test borings in several parts of the proposed site. Good wash borings will generally furnish this information, but core borings are more reliable though somewhat more expensive. A few years ago it was customary to drive test piles, in order to determine what length of pile to specify in the contract, but this plan was usually rather expensive, unless a pile-driver was on the site. In one instance four timber test piles cost $200 each, on account of the expense of moving the pile-driver to the site. Test borings should show the various strata, their thickness, etc., and the elevation of the ground-water; the strata will be of different thicknesses and elevations in different parts of the site, and the borings should provide the informa-

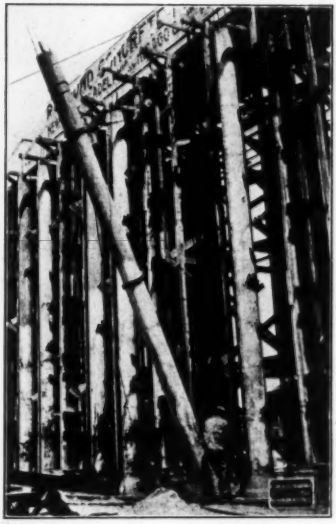


FIG. 1.—METHOD OF MOULDING AND LOWERING COMPLETED PILES, ATLANTIC CITY BOARDWALK.

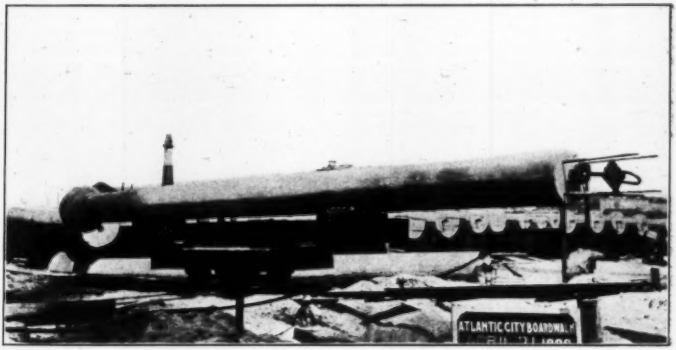


FIG. 2.—VERTICALLY MOULDED PILE, ATLANTIC CITY BOARDWALK.

tion necessary to determine the length of pile, the type of pile to be used, and on which stratum to place the load. Neglect of these precautions is responsible for some of the settlements which have occurred in pile foundations, both timber and concrete. One instance may be cited where a 10-ft. fill had been placed on a swampy piece of ground, the bottom of which was 40 ft. below. To have reached this hard bottom, the piles should have been 40 ft. long, but a tapering pile of about two-thirds that length was used, and, for its support, relied on the filled layer and the spread of the concrete capping, and thus made what might be termed a floating foundation. This foundation settled slightly at first, and continued to go down somewhat each year until the total settlement is now more than 3 in.

One advantage of made-in-place piles over moulded piles is that, as they harden in the place where they are to be used, a small foundation could be completed before the moulded piles would be ready to drive, thus effecting a small saving in time.

In the construction of the foundations of a garage at Albany, N. Y., an unpatented type of this class was used, which serves to show one of its possible variations. The original surface of the site was a stiff clay and had been covered to the depth of 15 ft. by clay from adjoining excavations; thus it was necessary to carry some of the load to the original surface. This was accomplished by excavating holes, 10 in. in diameter, through the overlying fill to the original surface, using a screw post-hole digger, and then filling the holes with fresh concrete, the clay filling being stiff enough to retain its position during the operation. The piles were spaced at 3-ft. centers, two staggered rows being placed under the walls, while the pier loads were supported by clusters similarly spaced. The total length of piles was 4 779 ft.; the labor of digging the holes cost 20 cents per lin. ft., and the concrete $7 per cu. yd. in place, or about 13 cents per lin. ft., making a total of 33 cents per lin. ft. of completed pile. In pile work there is usually quite a large charge for plant installation and dismantling, but in this instance this was reduced to a minimum, which accounts for the low unit cost. No compression of the pile hole was made, but the supporting power of the pile was assisted materially by the capping, which acted as a spread footing.

Another type of this class has been used in the clay soils around Boston, Mass., where the clay was overlaid by some 20 ft. of sand

and silt. Through this an 8-in. casing was forced into the clay and the materials were washed out. Then a chambering device was used to expand the footing of the pile by cutting away the clay, which was also removed by washing. In this way the area of the footing of the pile was made 16 times as great as that of the pile itself—a very effective method of obtaining a spread footing at considerable depth at a moderate cost. This type of pile has been patented.

Just previous to the construction of the pneumatic foundations of the Trinity-United States Realty Buildings, at Broadway, Cedar Street, and Trinity Place, New York City, the foundations of three columns of the Sixth Avenue Elevated Railroad, located on the curb line of the Trinity Place front, were carried to hardpan, 75 ft. below the curb, by using steel-encased concrete piles. This was done in order to avoid any possible settlement of the railroad structure during the sinking of the caissons, which were to be carried to bed-rock. The original piers of these columns were founded on the sand at about ground-water level, 12 ft. below the curb. Below this footing elevation there was a thick stratum of quicksand extending to the hardpan, the latter resting directly on the bed-rock. The transverse girder supported by the first column was shored up, and the column and its pier were removed and then restored completely before any work was done on the next column. Over the close-sheeted excavation made for the removal of the old pier there was used a short pile-driver which could be operated under the overhanging girder, and this was assisted by a water jet, to force a 12-in. standard, black, wrought-iron pipe, in random-length sections, to the hardpan. When the first section had been worked down to the bottom of the sheeted excavation, another section was joined to it with outside couplings, the threads on the ends of the pipes being cut long enough to allow the ends of the sections to butt properly. The lower end of the bottom section was protected in most cases by an open driving shoe, which made it easier to sink the pile shell than with a cone-shaped, closed point, as it allowed the free use of the jet to soften up the material. The upper end of each section was protected from the direct impact of the hammer by a driving head which screwed into its coupling and butted on the pipe. This driving head had a slotted opening in its side through which the jet was worked during the operation of the hammer. Most of the material from the inside of the pipe was removed by the

FIG. 1.—TYPE B, MADE-IN-PLACE CLASS: METHOD OF MAKING CASING.

FIG. 2.—TYPE B, MADE-IN-PLACE CLASS: PILE EXTENDED ABOVE GROUND TO RECEIVE CAPPING

jetting process; any that remained was taken out by an ordinary sand pump. There was no difficulty in getting out all the material after the pipe had reached the hardpan, the depth being checked by steel tape measurements, and the interior of the pipe inspected by the aid of a lantern. The pipe was then filled with 1:2:4 concrete. Five piles were thus placed for each pier, one near each corner, and the fifth in their midst directly beneath the center of the column. The piles extended 5 ft. into the mass of concrete which formed the new pier, and was 8 ft. square at the base and stepped in to meet the column base at the curb level.

A similar pile has been used in the underpinning of buildings in New York City. It is placed directly beneath the wall by cutting in the latter a vertical opening, from the starting elevation and through the foundation courses, of sufficient width and depth to leave ample working space around the pipe. The sections are forced into the soil by hydraulic jacks assisted with water jets, using the remainder of the wall for reaction purposes.

These piles are of various diameters, from 8 in. to 3 ft., in proportion to the loads to be carried, and the spaces are cleaned out in a manner similar to that just described, except that the largest sizes, in which there is space enough for a man to work, are cleaned by hand. The spaces are then filled with concrete under air pressure. When the pile is completed the load of the building is transferred to it by masonry and wedging stones. It is the most effective system of underpinning in use to-day, as it gives a positive assurance of the safety of the building, and at the same time does not occupy any space outside the wall, thus the contractor has a lot free from all obstructions.

Some of the tall buildings in New York City are supported on foundations of a patented type composed of steel-encased concrete piles, heavily reinforced throughout, and driven to place in a manner somewhat similar to that used in sinking other encased piles, except that a pneumatic hammer is used in connection with the water jet. The shell of this pile is generally $\frac{3}{8}$ in. in thickness and 12 in. in exterior diameter, the sections being joined by inside couplings. The reinforcing rods are inserted in sections, which are fastened together by an effective mechanical device, and form a continuous rod. The earthy material in the shell is withdrawn, and is replaced by concrete,

in the manner heretofore described. On account of the large area of steel in its cross-section, the Building Department of New York City has given this pile a much higher bearing value than any other, namely: 350 lb. per sq. in. on the concrete in compression and 4 200 lb. per sq. in. on the steel. The life of the steel in the exterior shell is somewhat doubtful, as it is subjected not only to the oxidizing influence of the ground-water, but also to the corrosive action of gas in the soil, and to stray electric currents; therefore it should not be considered as a permanent element in the strength of the pile.

The cost per linear foot of the made-in-place class varies with the type of pile, the amount of reinforcing, the size of the contract, the accessibility of the work, and the sub-surface conditions. As no two pieces of work are alike, it is difficult to give more than a general idea of the cost. In the representative types of this class the cost will vary from 75 cents to $1.50 per linear foot of pile, depending on the foregoing conditions; the steel-encased varieties are more expensive, as the casing alone costs more than the above-mentioned price.

Concrete piles of this class should always be reinforced in order to provide for all cross-strains due to sub-surface conditions or those which may take place during construction, and while the pile is still green, particularly when the length of the pile is more than 15 ft. Piles of this class are better adapted to lengths of less than 30 ft., while, for greater lengths, a moulded type is more desirable, as there is doubt as to the homogeneousness of the concrete when it is dropped 30 ft. and cannot be thoroughly spaded in place.

In the second class of concrete piles—the moulded type—the patented varieties, like those of the first class, are more generally known than the unpatented. They also differ from each other in shape, cross-section, and method of reinforcement. The moulded pile was first developed in Europe, and is used extensively there, and also in the United States, for piers, docks, and similar structures, for which it is specially adapted. It is also being used in the foundations of buildings and other structures where subsoil conditions are favorable, and especially where long piles are desired. Piles of this class, more than 60 ft. in length, have been successfully driven, pulled out, and re-driven several times without showing any signs of weakness. All moulded piles are reinforced to stand the rough handling to which they are subjected in transportation from the moulding yard to the

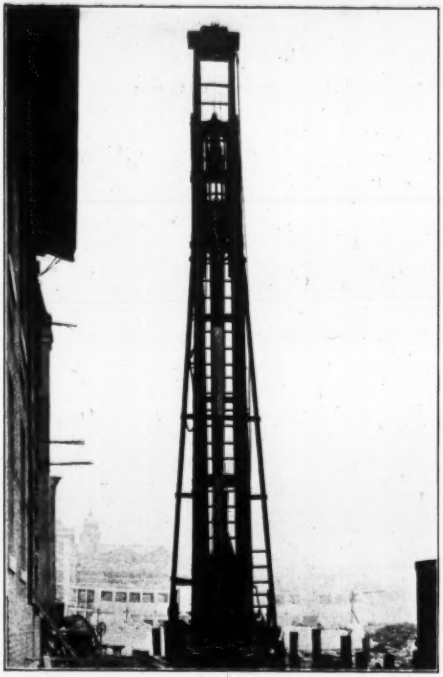


FIG. 1.—TYPE B, MADE-IN-PLACE CLASS: FORM ENCASED IN METAL, READY TO DRIVE.

FIG. 2.—TYPE 3, MOULDED CLASS: HAULING 61-FT. PILE INTO LEADS.

site and in being placed in the leads, as they are sometimes hauled up by a line attached to the butt of the pile, instead of by a bridle which would lessen the cross-strain. The driving strains are very great, and it is thought by many that a pile with reinforcement should not be subjected to the impact of a hammer; but many piles have been driven in this way, and later pulled up and re-driven, without showing any damage by the hammer. In one instance an attempt was made to destroy a pile by hard driving, but the only effect was to mash or broom the butt of the pile for about 2 ft., the remainder being uninjured.

Most of the moulded piles are driven into place; but, when conditions are favorable, they are jetted, and in some varieties special provision is made for jetting by inserting a 2-in. pipe in the center of the pile. In city work it is seldom practicable to use the jet, on account of danger to the foundations of adjoining property. When

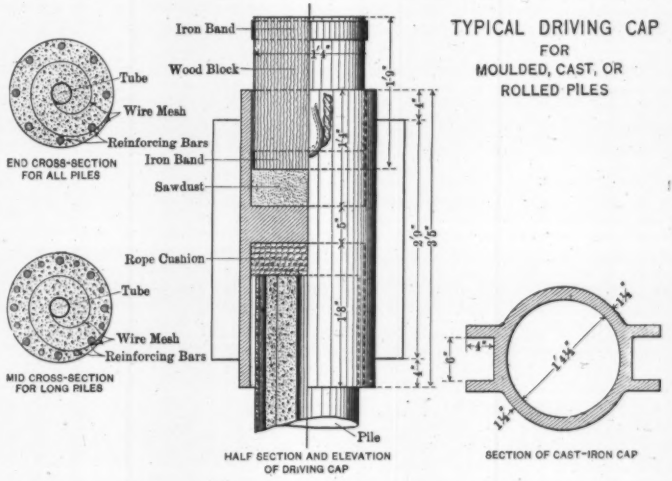


FIG. 1.

driven into place, some provision is made for cushioning the blow of the hammer. This is usually done by the insertion of a false pile of wood, acting on a sawdust or sand cushion, between the hammer and the head of the pile, the whole driving head being held in position on the pile by an encircling metal cylinder held securely in place.

Piles of this class are moulded either horizontally or vertically,

one variety being rolled instead of moulded or cast. The vertical system of moulding is the more expensive in time and labor, but, when thoroughly done, it produces a good pile. It is possible to effect considerable saving in forms alone by the horizontal system, as the upper portion or lid of the form for square or polygonal cross-sections can be omitted entirely, while the side pieces can be removed after 24 hours and used again, thus leaving only one portion—the bottom piece—tied up by each pile. In the vertical method all the sides of the form for the pile must remain in place until the pile is strong enough to stand alone. On a large installation quite a sum of money can thus be saved on forms, and, naturally, the horizontal method is the more popular.

One of the best known types of this class, and among the first to be invented and practically applied, is a straight pile of square cross-section with the corners slightly chamfered. It is reinforced with ¾-in. rods, one near each corner of the square, and connected with each other at intervals of 12 in. by diagonal and straight wire ties, the lower end of the pile being protected with a pyramidal metal shoe firmly attached to the reinforcement. Provision for jetting is made, in the lower end only, by the insertion of a piece of wrought-iron pipe, about 3 ft. long bent in such a manner as to extend from the center of the point to the outside of the pile just above the shoe, where an outside connection can be made and then removed, thus making it possible to save a considerable length of pipe and at the same time make a more solid pile. This type has been used in all kinds of foundation work, both in the United States and abroad, particularly in structures along water fronts, bridges, buildings, etc.

Another well-known type of the moulded pile is tapering in shape, 16 in. in diameter at the butt and 11 in. at the point. It is octagonal in cross-section, each of the faces of the octagon containing a semi-circular longitudinal corrugation for almost its entire length. It is reinforced throughout with Clinton woven-wire cloth, and a central opening, for jetting purposes, is moulded in the concrete by the use of a tapering collapsible mould, thus obtaining an opening for the water jet at small expense. After the pile has been placed, this opening can be filled with grout and fine concrete, thus making a solid pile. A driving-head, somewhat similar to that previously described, is used with this pile when it is driven into place by impact. It is claimed for this pile that, as the corrugations increase its superficial

FIG. 1.—TYPE B, MADE-IN-PLACE CLASS: FOUNDATION READY FOR CAPPING.

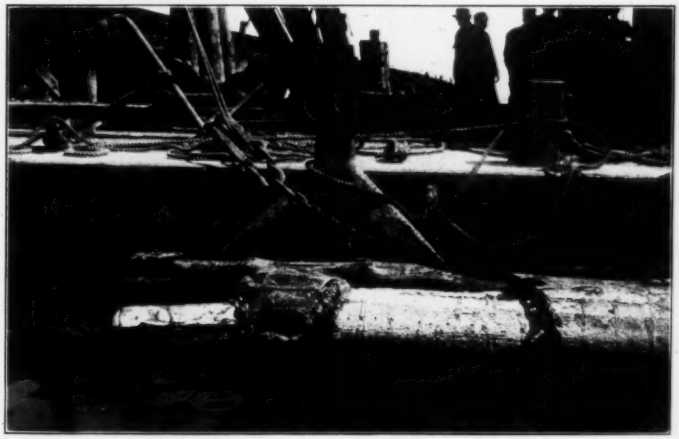


FIG. 2.—TYPE 3, MOULDED CLASS: EXAMPLE OF PILE DRIVEN TO REFUSAL SEVERAL TIMES.

area, its bearing capacity is increased correspondingly, and the corrugations assist materially in the escape of the water, making it more easily put in position.

A third type, circular in cross-section, is formed in an entirely different manner, in that it is not moulded or cast but is rolled mechanically into shape. A sheet of ordinary black wire fabric, of No. 16 gauge, with ½-in. mesh, and of the length required to make the pile, is attached to the rod which serves as the winding core and later as a part of the reinforcement of the pile, by small rivets driven into holes slightly smaller. The other edge of the sheet is attached to a movable platform which is pulled toward the core as it is wound up. On this sheet is placed the other reinforcement, and the whole is covered with a layer of fine concrete. The mass is then wound up like a jelly roll, and fastened with wire ties every 6 in., and more frequently where needed. In some of the later piles the method of fastening the outer edge of the wire cloth has been changed by omitting the wire ties and substituting a rivet with its end upset so that it is slightly larger in diameter than the mesh of the cloth. These rivets are inserted in the outer edge of the cloth with this upset end upward, and, as the pile is rolled up, they are forced into the meshes of the fabric in the body of the pile, thus tying or rather buttoning it together. The platform on which the pile is moulded is inclined, so that, without any lifting, the completed pile is rolled therefrom to the car which is to carry it to the seasoning yard. The platform of the car is also inclined, and, when the car has reached the yard, the pile is rolled to its place on skids. By this method no large plant, except the rolling device, is required to handle the pile during its formative stage. When these piles are to be driven into place, a driving head is used, and when jetted, a wrought-iron jetting pipe forms the winding core. Piles of this kind, 61 ft. long, have been driven, pulled out for examination, found to be intact, and re-driven. Like other piles of this class, they have stood very severe treatment. In one instance an attempt was made to destroy a pile by over-driving; later the pile was pulled up and the only apparent injury was the mashing or brooming of the upper 2 ft. This type has been used in the foundations of some of the stations of the Brighton Beach Railroad, in Brooklyn, N. Y., in filled ground. It has also been used in other places throughout the United States.

At Kansas City, Mo., solid octagonal piles (unpatented) were

used in the foundations of the Sixth Street Viaduct. These piles were moulded horizontally, and, when given sufficient time to harden, were successfully placed. The contract was let in the fall, and as it was found to be impossible to mould enough piles to complete the work, it became necessary to use some creosoted piles.

Moulded concrete piles were used during 1908 in the construction of a boardwalk, 50 ft. wide, at Long Beach, Long Island. These piles were moulded horizontally, and were 11 in. square at the butt and 8 in. square at the point, with the edges chamfered; they were reinforced with a 1-in. bar in the center. The piles were 25 ft. long and were jetted 15 ft. into the sand by the use of a twin outside jetting apparatus which worked very successfully.

In the construction of a pier at Atlantic City, a few years ago, there was used a moulded concrete pile with a bulbous end of much larger diameter than the body of the pile. These piles were moulded on end and jetted into place. The pile was 25 in. in diameter, and the bulb at its greatest diameter was 3 ft. 6 in. This pile was reinforced with eight $\frac{3}{4}$-in. bars symmetrically placed around a $2\frac{1}{2}$-in. jet pipe in the center.

During 1908 a moulded pile of the same general style was used in the construction of a boardwalk at Atlantic City, N. J.

In 1906 the Atlantic and Birmingham Railroad built two piers at Brunswick, Ga., on moulded piles somewhat like the pile of this class which was described first. The piles were 16 in. square, and were straight, except that the lower 10 ft. tapered to 8 in. square at the point, and the edges were chamfered. The top of each pile, for 22 in., was 10 by 16 in. to allow of a bolted connection with the wooden girders of the superstructure. They were reinforced with four $1\frac{3}{8}$-in. rods held in position at 12-in. intervals with wire clips, and fastened to the jet pipe at the point. These rods were extended to within 3 ft. of the top of the pile, the reinforcement in this part being woven-wire fabric. About 1 500 of these piles, from 32 to 50 ft. long, were installed. One would naturally expect that timber piles would have been used, as Brunswick is the shipping port for yellow pine and its products, but it was not practicable to use timber on account of the marine borers. The supports of the timber docks now in use require constant repairs, and the timber pile does not last long enough to pay for its installation, even though it may be very cheap. On the

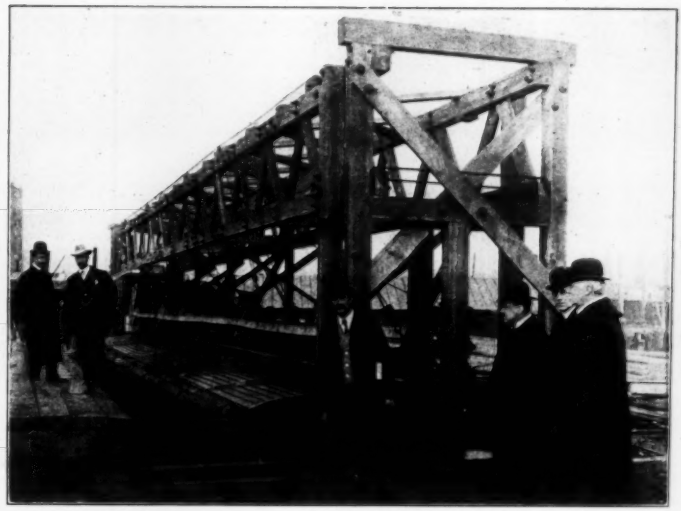


FIG. 1.—TYPE 3, MOULDED CLASS: MACHINE FOR ROLLING THE PILE.

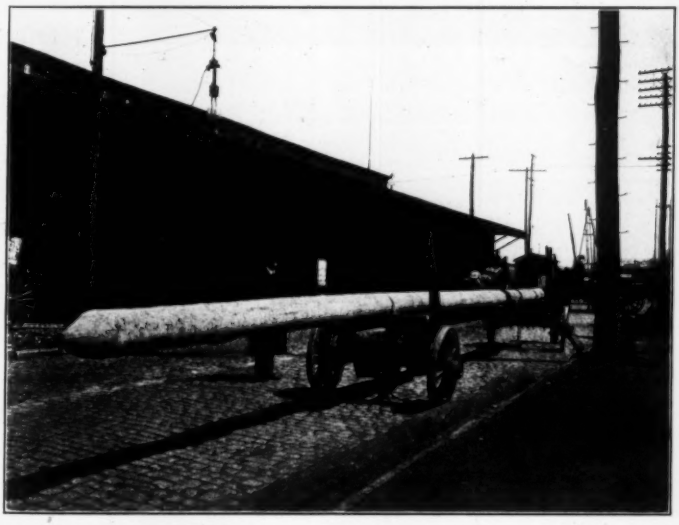


FIG. 2.—TYPE 3, MOULDED CLASS: PILE 14 IN. DIAMETER; 50 FT. LONG; WEIGHT, 3½ TONS. HAULED 4 MILES TO SITE OF WORK.

other hand, these new concrete piles make a permanent pier and will never require replacing.

In a power-house foundation at Bridgeport, Conn., a composite timber and concrete pile was used. The site was on the banks of a tidal stream where the muck deposit was very deep, and this pile was used in order to save the expense of excavation and concrete footings. The timber pile was driven about flush with the surface of the ground, then a hollow, steel-pipe follower, 12 in. in diameter, was used to drive it below water level. The follower was then filled with concrete and withdrawn, leaving the concrete in place on the butt of the pile. A mechanical bond between the timber and the concrete was obtained by the use of long metal dowels which were driven into the butt of the timber pile before the follower was placed in position.

Moulded concrete piles, of rectangular cross-section, and having a male and female joint on two edges, have been used as sheet-piles in many cases during the past few years. One type is 6 by 14 in. in cross-section, with a reinforcing rod near each corner, the rods being held in position by wire clips at regular intervals. These sheet-piles are installed in a manner similar to that heretofore described, either by jetting or driving, or a combination of both, as may be desirable.

There are in use to-day many other varieties of concrete piles which differ in some respect from those described, either in the reinforcing, the shape, or the method of installation, but their general characteristics will place them in one of these classes.

The different shapes and sections of the various concrete piles have led to considerable discussion between their respective advocates as to the relative bearing power of each pile, most of the contention referring to the shape of the pile and whether the tapering or the straight shape (in which the pile is of uniform cross-section throughout) has the greater supporting value.

The tapering pile depends for its bearing power more on its wedge shape and the skin friction thus developed than on its efficiency as a column, while the straight pile relies on the skin friction developed by its exterior surface, and also on its strength as a column. The amount of skin friction developed by any pile depends, not only on the character of its surface, but also on the area of that surface. Considering for the present only the area, and assuming two piles of equal length and having butts of the same size, one tapering to

6 in. at the point while the other is straight throughout, it will be found that the straight pile has a larger surface area on which to rely for skin friction, and has a larger bearing surface at the point. If these piles are each 14 in. at the butt and 20 ft. long, the tapering pile will have a surface area of 52.36 sq. ft. as compared with 73.30 sq. ft. for the straight pile; while the point area of the tapering pile will be 0.196 sq. ft. and that of the straight pile 1.069 sq. ft., the latter having almost 5½ times as much area available to transmit its load as a column, about 1.4 times as much superficial area, and 1.7 times as much volume, which also represents its displacement.

The relative compressions may be seen by Table 1.

TABLE 1.

Type of Pile.	Diameter, in Inches.		Circumference, in Feet.		Cross-Section Area, in Square Feet.		Surface Area, in Square Feet.	Volume, in Cubic Feet.
	Butt.	Point.	Butt.	Point.	Butt.	Point.		
Tapering.	14	6	3.665	1.571	1.069	0.196	52.36	12.64
Straight..	14	14	3.665	3.665	1.069	1.069	73.30	21.42

Comparing the two piles again, for the purpose of ascertaining the relative frictional values of their upper and lower parts, it will be found that the tapering pile has a much larger proportion of its surface area in the upper than in the lower zone, which indicates that it must depend on the upper layers of the ground for the greater part of its supporting power, while the straight pile has its surface area uniformly distributed and can utilize all the strata through which it passes.

It is apparent from Table 2 that 32.5% of the surface of the tapering pile is in its upper quarter while only 17.5% is in the lower quarter. The Building Department of New York City allows 15 tons per sq. ft. on rock and 10 tons per sq. ft. on firm gravel or clay, when foundations are carried down through earth by piers of stone, brick, or concrete; on this basis, assuming a load of 20 tons on each pile, and that each has been driven to the firm gravel, so that it can act as a column, the tapering pile, with its 0.196 sq. ft. of area at the point can transmit as a column only 1.96 tons, leaving the remainder of the load, 18.04

tons, to be supported by skin-friction, and about one-third of this would have to be supported by the upper stratum of the ground. The straight pile, having 1.069 sq. ft. of area at the point, can sustain as a column 10.69 tons, leaving 9.31 tons to be carried by skin friction, and about one-quarter of this, or 2⅓ tons, would be supported by the upper stratum of the ground.

TABLE 2.

	Circumferences, in Feet.		Superficial Areas of Zones, in Square Feet.		Percentage of Zone Areas to Entire Pile.	
	Tapering.	Straight.	Tapering.	Straight.	Tapering.	Straight.
Butt	3.665	3.665	17.020	18.325	32.5%	25%
One-quarter	3.142	3.665	14.400	18.325	27.5%	25%
One-half	2.618	3.665	11.780	18.325	22.5%	25%
Three-quarters	2.094	3.665	9.160	18.325	17.5%	25%
Point	1.571	3.665				
			52.36	73.30	100.0%	100%

The basic reason for the use of piles for foundations is that the surface ground at the site has not sufficient stability to support the load; therefore piles are used as a means by which to transmit it to a lower and more substantial stratum. Structures supported on tapering piles, in districts subject to floods and the scour of currents, are likely to settle seriously if erosion takes place, and, under these conditions, the piles should be driven to a firm bearing stratum and not rely on the upper one to support the load.

The supporting power of a pile due to frictional resistance depends largely on whether the surface of the pile is smooth or rough, thus a rough-surfaced concrete pile will have more sustaining power, due to that element, than a pile of the same size with a smooth metallic or wooden surface. The supporting power due to the same factor also depends on the material in which it is driven, which may be anything from a fluid muck to a solid gravel or clay; it is possible to have any of these features in combination with those above mentioned, therefore it is important to study carefully all the conditions of each problem before deciding which type of pile to adopt.

DISCUSSION.

Mr. Tower. MORTON L. TOWER, M. AM. SOC. C. E. (by letter).—Owing to the continual rise in the price of timber, as well as on account of the indestructibility of concrete piles, the latter are sure to have an increasing range of application. Even in the locality of San Francisco, with the fine Pacific Coast pile timber within a radius covered by low ocean freight rates, the concrete pile has found a demand, and its design is a matter of interest and a subject of study for engineers.

For lengths of 60 ft. and less, the design of reinforced concrete piles contains no unusual features, and numerous examples have been illustrated and described in the technical magazines and society proceedings. For piles more than 60 ft. long, the weight and strains due to handling render excessive reinforcement necessary, and, thus far, such piles have had few, if any, practical applications. In all probability, a long pile could be designed in the form of a hollow concrete cylinder with longitudinal diaphragms, which would be considerably lighter than a solid pile.

Metal points on "made-before-driving" piles seem to be superfluous. Shoes were once deemed essential on wooden piles, but are seldom used at the present time. For the "made-in-place" type, using some form of light metal casing driven into the ground, the metal point is of course essential.

Piles with square ends are much more easily kept in line while driving, while the points add little if any to the rate of penetration. The advantages of a square end for bearing power are very evident.

For these reasons, concrete piles for general use should be designed with square ends; for stiff clay soils, resulting in hard driving throughout the entire penetration, the pointed end may be of sufficient advantage to warrant its use.

The advantages of the water-jet for placing concrete piles are very evident. Probably this method of driving has been developed more fully at the jetty at the mouth of the Columbia River than anywhere else.

The details of the apparatus and methods used there, with very few changes, would apply to the handling and placing of concrete piles. Briefly, this consists of two jets, one on each side of the pile, the water emerging from 2½-in. pipes with no end contraction, the lower end of the pipes projecting from 3 to 4 in. below the lower end of the pile. After the pile enters the ground, the water pressure is maintained at from 80 to 120 lb. A duplex pump, 10 in. in diameter and with an 8-in. stroke, is used, the boiler pressure being from 100 to 120 lb. Between the pump and the pile the water is carried by two lines of 4½-in. pipe and fire hose. Piles with points not less than 10 in. in diameter have been driven to a penetration of 30 ft. in from

10 to 15 min., including the time required for securing the grade, as they are not sawed off. Mr. Tower.

In numerous contractors' plants the boiler and pump capacity are too small to furnish a large volume of water, and the writer believes that failure to recognize the advantages of the larger plant has been the reason why the jet method has not as yet come into more general use.

The jet method is applicable in any soil which will settle about the pile after the flow of water is stopped. For pure ocean or river sand, it is the best method to adopt.

One great advantage of the jet is that there is absolutely no danger of ruining the pile by over-driving, as shown by the illustrations in Mr. Cole's paper. It is the writer's opinion that piles are very often driven to destruction, under the direction of conscientious but inexperienced inspectors.

As far as the writer is aware, there is no formula for computing the bearing power of piles jetted to place, and, for the bearing capacity, one would have to depend on comparison with driven piles, or on actual tests on piles in similar soils.

The condition of the pile surface, as to roughness or smoothness, cannot have any great effect on the bearing capacity. Small indentures become filled with compressed soil, which, as far as resistance to settlement is concerned, become a portion of the pile; and the frictional resistance to the pile moving through the soil changes to a shear in the soil over the small area in question; the resulting difference in bearing power would be of no practical value.

L. J. MENSCH, M. AM. SOC. C. E. (by letter).—Mr. Cole's paper treats of a subject which deserves serious attention. Very often expensive foundation methods are resorted to where the work could have been done with reinforced concrete piles for less money and in less time. Mr. Mensch.

It is the writer's opinion that Raymond or Simplex piles should only be used for short lengths and where the strata penetrated are well-known from previous test-borings, and consist of easily compressible soils containing very little ground-water. The driving of the steel pile compresses the soil, and there is a tendency to diminish the size of the hole when the pile is withdrawn, which cannot be prevented by the loosely connected, No. 20, sheet-metal shell of the Raymond pile, or by the weight of a few feet of concrete, in the case of the Simplex pile. There is much less danger in this respect if the hole is drilled by the ordinary well-driving process and sections of steel pipes are sunk, which are afterward withdrawn, and the hole filled with concrete similar to Simplex piles. The drilling does not compress the soil, and, therefore, there is less tendency to close the hole.

Mr. Mensch. In all made-in-place piles there is considerable danger of obtaining a poor quality of concrete, when dropping it for considerable distances, and a very careful supervision is absolutely necessary in order to obtain reliable results.

When driving Raymond or Simplex piles there exists a danger which has caused failure several times, in that the adjacent green piles are injured by displacement. Ground-water, also, has often caused failures in such piles.

Piles, cast in holes made by the well-driving process, can be used with great advantage where only a few are needed and where it does not pay to install a pile-driving plant, or inside of buildings, or in underpinning buildings, where the driving of piles might injure existing foundations.

Mr. Cole mentions some cases where the steel pipes remained in place. Inasmuch as it is not known how long the pipe may last, it cannot be said that the pile is strengthened by the shell, and, in most cases, the use of such piles represents a waste of money, and a fear on the part of the designer to use reinforced concrete piles moulded in advance.

In France made-in-place piles are often made by the Compressol method. A hole is made in the ground by letting special hammers weighing about 2 tons fall from a height of 50 ft., and then strengthening the wall of the hole by tamping in clay with a hammer of a special shape. Unless great precautions are taken in concreting, the dirt of the wall may be entrained in pouring and a poor pile obtained. The fact that made-in-place piles are often shown to carry considerable loads under tests can be explained in most cases by the friction of the pile against the upper hard crust which generally overlies the yielding stratum. Tests made by the writer have shown this friction to amount to as much as 1½ tons per sq. ft. of contact surface in good fill. In such cases a shorter pile will do the same work as a longer pile, the greater part of which, being in mud, does not add to the carrying capacity.

In the writer's opinion there are no good reasons why made-in-place piles should be adopted in preference to moulded piles, the latter type being undoubtedly the safer.

Under good management it is possible to build moulded piles for practically the same price as other piles, and in regard to speed of construction, the writer has built piles at the rate of 40 per day and has driven 24 per day, and this is not the limit of speed.

There is no patent on the principle of reinforced concrete piles, most of the patents relating to minor details which are a detriment to, rather than an improvement on, common-sense construction. Concrete piles need only sufficient reinforcement for safe handling before driving. When they are well made and a good driving-cap is used,

there is less chance of such a pile breaking than is the case with a timber pile. Concrete piles should have rather a blunt point, or may be cut off square, and, in case the driving is done by a water jet, it is advisable to use a concave point when only one central jet is used. To the writer's knowledge, the longest concrete piles ever driven were 75 ft. long and 16 in. square, reinforced by four 2-in. round bars. Mr. Mensch.

These long piles are very expensive, however, and can be supplanted by a composition pile, such as is used in the wharves and sea walls of San Francisco Harbor. A wooden pile from 50 to 60 ft. long is first driven so that the top projects about 10 ft. above the mud-line, which is from 20 to 30 ft. below the top of the wharf. Over this wooden pile, a hollow concrete pile from 2 to 3 in. thick and having an outside diameter of about 24 in. is driven to a good bearing in the mud. These hollow piles are reinforced by from 4 to 6 sq. in. of steel in a longitudinal direction and by No. 1 wire, 4 in. on centers, in a circumferential direction. The mud inside the pile is ejected by a water jet, and sometimes the water is pumped out, and the interior filled with concrete. The advantage of hollow piles over concrete piles lies in the lower cost of the composition pile and also in the greater stiffness. Besides, the bearing of the 24-in. pile adds considerably to the sustaining power of the composite pile. Such composite piles can also be used with great advantage on land, and are certainly cheaper than long concrete piles.

The writer does not think that Mr. Cole's comparison of a 14 by 14-in. straight pile with a tapered pile 14 in. at the butt and 6 in. at the point, is fair. If he had compared a tapered pile having the same amount of material as the straight 14 by 14-in. pile, which tapered pile would be 22 in. at the butt and 6 in. at the point, he would have obtained the same surface in both cases, and there is then left in favor of the tapered pile the very considerable wedge action on the ground.

CHARLES H. HIGGINS, ASSOC. M. AM. SOC. C. E. (by letter).— Mr. Higgins. Mr. Cole, in his interesting paper on "Concrete Piles," brings forward a matter of great economic importance.

In his preliminary remarks on wooden piles, he cites instances of such piles, exposed by subsequent excavation, that were "buckled and crimped into accordion-like folds, and had no sustaining power whatever." Undoubtedly, many wooden piles are injured in driving, particularly when the purpose is to drive to a certain depth without regard to the resistance, as in the case referred to. The author states that:

"In the construction of caisson foundations for sky-scrapers in New York City, it has been the custom in many instances to drive timber piles to form a temporary support for the derrick traveler. These piles remain in place until all material has been excavated."

Mr. Higgins.

Probably, if the purpose had been the ordinary one of obtaining resistance to carry a load, driving would have ceased before the piles got into the condition shown, as all who are experienced in such work realize the danger from over-driving; but, even if a pile be damaged in driving, if it is true that the resistance to the last few blows of the hammer measures the carrying capacity of the pile, it can hardly be said that it "has no sustaining power whatever."

Mr. Cole's remarks on timber piles in piers, docks, and similar structures will be accepted by most engineers, but, what is the life of a concrete pile alternately covered and exposed by the waves and tide in this climate? If, under these conditions, the surfaces of some bridge piers and concrete bulkheads deteriorate, the position of a pile or column, not more than 20 in. in diameter, exposed on all sides to the waves and tide, and loaded to several hundred pounds per square inch of section, is one not without hazard.

It is not quite clear to the writer what is meant by "comparable size," near the bottom of page 469. If two piles, one of concrete and the other of wood, have the same diameter at similar places in their length, and the same volume, one would hardly expect the concrete pile to carry double the load of the wooden pile, unless it was heavily reinforced with steel. In such a pile, the steel becomes as great a factor in the cost as the concrete.

In the comparison of the relative cost of concrete and timber pile foundations at Annapolis, Md., cited by the author, the length of the piles is not stated, but the price, $9.50 for wooden piles, in such a large quantity, would indicate a length of not less than 50 ft. As 2 193 wooden piles were to be used, or 855 concrete piles, if it be assumed that 12 tons was the working load for the former, 31 tons would be the load of the latter. Ordinarily, a concrete pile, 50 ft. long and capable of bearing safely 31 tons, would certainly cost not less than $1.20 per lin. ft., or $60 each, and 855 piles at $60 = $51 300, instead of the $17 100 given in the text, or a total of $59 603.20 for the foundation with concrete piles, as against $52 859.18 given as the cost of the foundation with wooden piles. It would be interesting to know what conditions made the estimate quoted possible.

In regard to the made-in-place piles referred to as Class A, the writer has seen such piles driven, and, under certain subsoil conditions, it would seem to be an excellent method; but, remembering how few excavations made in soil which is usually piled do not lay bare unexpected conditions, he would not feel secure in their use.

Taper may be an excellent quality in a pile under certain conditions, but, in the main, the purpose of piling is to transmit the load through a soft stratum to a hard one into which the piles usually do not penetrate far, and, for this purpose, a large area at the point and stiffness as a column would seem to be the chief requisites. The

reason wooden piles have a taper is obvious, but the same reason does not obtain when concrete is used. Mr. Higgins.

The behavior of the piles referred to as Type 3, shown on Plate XXXIV, Figs. 1 and 2, Plate XXXV, Fig. 2, and Plate XXXVI, Figs. 1 and 2, was carefully watched by the writer during the making, handling, and driving. They were handled in the manner customary with wooden piles, and required no greater care. The pile shown on Plate XXXV, Fig. 2, after being driven in sand, with a hammer weighing 3 500 lb., to an extent that would have crippled a wooden pile, showed no damage on being withdrawn, except for about 2½ ft. of the head, which is shown in the photograph. Aside from the cost, it would be hard to find any objection to them. How they will stand the action of sea water when alternately covered and exposed by the tide and beaten by the waves during freezing weather is a question the writer is trying to determine by observation of one made at the same time as those shown on the plates, and put in position about a year ago. It is still too early to draw any conclusions.

Excellent concrete piles can be made, but their use, except under exceptional conditions, is prohibited by their cost. The reference to a composite wood and concrete pile used at Bridgeport is of interest, as it is along this line that the writer believes development will occur. Wooden piles are spliced successfully with a dowel and ferrule. The writer has in mind several important structures carried by such piles, where, due to lack of head room, 40-ft. or 50-ft. piles were driven in three or four sections. Therefore, as the chief advantage in the use of concrete piles for foundations occurs when the permanent ground-water level is a number of feet below the surface, as pointed out by Mr. Cole, why cannot a section of cast concrete pile, properly reinforced, be spliced to a wooden pile, thus gaining the advantage of permanency in the concrete above water, and, at the same time, reducing the total cost of the pile by using wood below water where it is equally permanent?

In the case of a 50-ft. pile, where the water level is 10 ft. below the surface, the cost of a concrete pile at $1.20 per lin. ft. = $60, the cost of a composite pile would be:

Wood, 40 lin. ft. at 20 cents per lin. ft.	= $8.00
Concrete, 12 lin. ft. at $1.20 per lin. ft.	= 14.40
Dowel and ferrule	= 5.00
	$27.40

or a saving of $32.61 per pile, over an all-concrete pile, and a saving in excavation and concrete piers over wooden piles cut off at water level.

Mr. Upson. MAXWELL M. UPSON, ASSOC. M. AM. SOC. C. E.—It is to be regretted that more definite data on the carrying capacity of piles of various shapes and types are not available. The speaker has examined all the available literature on this type of foundation construction, and has found comparatively little that can be used as a direct guide in predetermining results.

The chief reason for this is that the solution of a piling problem depends entirely on the nature of the soil, and, as all engineers know, there are almost as many kinds of soil and combinations of strata as there are piling jobs. For that reason the engineer who has had the most experience in the design and construction of pile foundations is most loath to place himself in writing on its theories and laws. Because of this lack of information, the speaker has been accumulating data for the past two years, with the object of eventually putting it into a paper to be read before this Society. These data are not yet complete, and it is only with a view of calling attention to information which seems to be at variance with statements in Mr. Cole's paper that the following is submitted.

The speaker desires to refer particularly to the relative advantages of tapered and straight piles. During the past two years it has been his privilege to have charge of the driving of a great many thousand concrete piles, comprising more than 75% of those driven in the United States. Within that time, soil conditions have been encountered in about a hundred different locations. On this information, together with carefully kept records of pile-driving for the last ten years, the following conclusions are based.

The speaker has not attempted to theorize on this information, or to draw up any theoretical tables to look well on paper and be used as an aid to enthusiastic but not always accurate salesmen. He admits that, during his early connection with work of this kind, he made attempts to do so, but found, however, that figures of this character were not borne out in practice, and that every rule seemed to have an infinite number of exceptions.

Mr. Cole seems to be under a misapprehension in regard to the "Type B" pile, as he has classified it only as a "tapered pile." It is not essentially a tapered pile, and, in fact, in several instances, where it seemed advantageous to use a straight or very slightly tapered pile, piles of this shape have been used successfully.

The inventor of this system (prior to its development) spent about 20 years as a wood-piling contractor, and built many of the railroad bridges in Kansas and Nebraska. It was his observation that tapering wooden piles in friction soil gave the better carrying capacity. On that account, when he invented the "Type B" system, known as the "Raymond," he experimented with a variety of tapers. His original design made it essential that the piles should have some taper in order

to keep the shell from creeping up on the core. Since that time certain changes and variations have been evolved which obviate the necessity of a tapering form and make it possible to use a perfectly straight core. The company which puts in this "Type B" pile is prepared to put in either straight or tapered piles. Mr. Upson.

The fact that 90% of the work executed by this company is based on tapered piles, indicates its firm conviction that this particular form has advantage in carrying capacity. This conviction is arrived at by information received from two sources: First, from a careful record of the resistance encountered in driving; and second, from a careful series of loading tests.

The method of placing these piles consists of driving, with a steam hammer, a steel core encased in a shell, which makes it possible to observe the penetration under each blow of the hammer; and since the weight of the core and hammer is constant, a comparison of the carrying capacity of each pile can be predetermined very accurately by noting the final resistance to each blow of the hammer, or, in pile-driving terms, the number of blows to the last inch. As a result of a large number of tests, it has been found that this method of measuring the resistance is practically as good as weighting the pile.

Loading tests have been carried on in which the real carrying power of the pile has been checked with the resistance demonstrated at the time of driving.

The following are the results of tests made in Boston in the fall of 1906, on three piles having a variety of tapers: These tests were conducted under the supervision of a prominent engineer who contended that, for the soil in question, a straight pile had advantages over a tapered pile, taking an attitude similar to that expressed by Mr. Cole, viz., that increased surface area afforded increased friction and consequently increased carrying capacity. To make sure of this point, two piles, each 20 ft. long, were driven within a few feet of one another. One core, known as "Core *C*," was 13 in. at the point and 18 in. at the butt, and the other, "Core *A*," was 6 in. at the point and 20 in. at the butt. It should be observed that it required 944 blows of a steam hammer to drive Core *C* 20 ft., the last inch of penetration requiring 8 blows. Core *A*, with the increased taper, required 875 blows to drive it 20 ft. into the ground, the last inch of penetration requiring 21 blows.

At the expiration of about one month, both these piles were carefully tested, the result of which is shown in Fig. 2. Pile *C*, with the slight taper, showed an initial settlement of $\frac{3}{4}$ in. with a load of 22 tons, while Pile *A* did not show a settlement of $\frac{3}{4}$ in. until after 63 tons had been placed upon it; in fact, it showed no settlement whatever under a load of 35 tons, and an inappreciable settlement up to 63 tons. These results are particularly interesting in view of the fact

Mr. Upson. that the engineer who made the tests did so for the specific purpose of demonstrating that a straight pile has increased carrying capacity. The results of these tests have been demonstrated again and again in the experience of the company with which the speaker is connected.

It is his firm conviction that the success of the work undertaken by this company, with an experience of eight years, in which no settlements have ever been encountered, and covering much more than half of the concrete piles driven in the United States, is due, first, to the absolute test which is afforded by watching the penetration of the pile

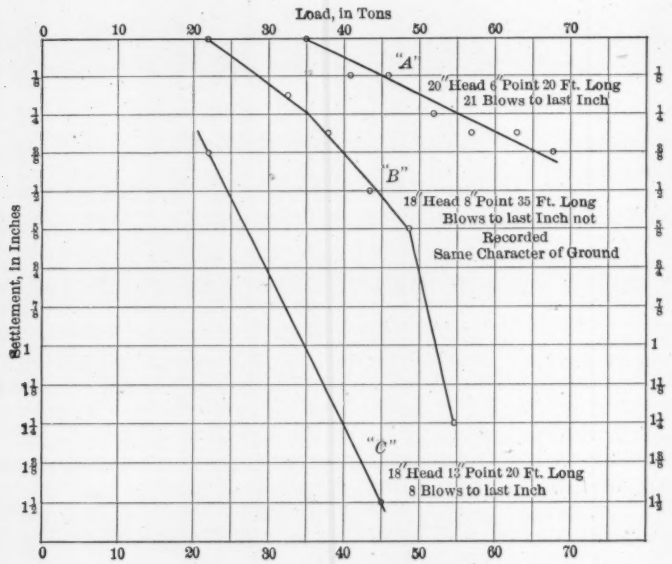


FIG. 2

while being driven; second, to the assurance that each pile is perfect, because the shell permits careful inspection; and third, because of the taper of the pile, which, in the speaker's opinion, increases the friction in proportion to the load imposed.

It is not contended that there are not many places in which a straight pile has its advantages. As a matter of fact, it is necessary where the top soil has little or no bearing capacity or may be washed away, and where the underlying strata are either hardpan, gravel, or rock. It is surprising, however, how infrequently these conditions are met in piling propositions.

Mr. Upson.

It does not seem wise at this time to attempt to enter into a discussion of the theory of why a tapered pile has a greater carrying capacity than a straight pile. There are several theories, and the speaker believes that they are obvious to most deep-thinking engineers. At a later date he hopes to present his views on this subject to the Society.

As a matter of economic interest, attention is called to the record (on Fig. 2) of the test of a 35-ft. pile driven in the same ground; this pile measured 8 in. at the point and 18 in. at the butt. It will be observed that on account of its decreased taper it was possible to secure a penetration some 15 ft. more than with the 20-ft. pile having the maximum taper; but, notwithstanding the increased length, its carrying capacity under a load was not as great as that of the short pile.

The failure to recognize this feature in piling has caused a material increase in the cost of many concrete pile foundations, for the reason that contracts are usually awarded on the basis of the lowest price per linear foot. It is well-known that, in a measure, the cost of piling is inversely proportional to the number of feet involved in the contract. It is obvious, therefore, that the bidder who contemplates the use of a straight or nearly straight pile or form realizes, unless the site overlies hardpan or rock, that any desired penetration may be secured. In other words, a straight pile, 40 ft. long, can be used where a tapered pile, 20 ft. long, would have been more effective. This means that twice the necessary linear feet of piling has to be paid for. This principle is well illustrated in New Orleans, where some recent important structures are resting on 20-ft. tapered concrete piles which were substituted for 70-ft. wooden piles. Tests on the short tapered concrete piles showed an increased carrying capacity of from 25 to 50% over the 70-ft. wooden piles.

The relationship between the number of blows per inch and the carrying capacity under test is well illustrated in the curves on Fig. 2. It will be observed that the pile with the slight taper brought up at a resistance of 8 blows per inch, while the pile with the increased taper required 20 blows per inch. The first pile showed a settlement of $\frac{3}{4}$ in. under 22 tons, and the tapered pile a settlement of $\frac{3}{4}$ in. under 65 tons. The ratio between the carrying capacity and the number of blows per inch practically agreed.

It is to be regretted that the method of checking the carrying capacity of piles by their penetration cannot be worked out in a built-up pile. This is due to two reasons: First, their mass is variable, and is so great that it is difficult to secure uniform conditions; and second, in order to preserve the pile, an elaborate system of cushions has to be used, the efficiency of which may vary almost 100 per cent. In other words, when a cushion block is new and in good working order, it may

Mr. Upson. transmit 75% of the blow to the pile, and when it is old or out of order it may absorb practically all the blow. This is a feature which is sometimes not considered by engineers who keep a record of the driving of built-up piles.

It has been well brought out by Mr. Cole that there is serious objection to the use of a permanent shell, on account of its cost. With this in mind, the speaker has made a series of investigations and experiments with a view of eliminating this from the "made-in-place type" of pile. As yet it must be admitted that a satisfactory way of obviating this expense has not been found. The more the matter is studied, the more difficult it seems. Piles must be driven in succession, within a comparatively short distance of one another, and for this reason it is found that, unless a shell or protection of some kind remains around the soft concrete, the driving of adjacent piles compresses the soil and cuts off or seriously warps and displaces the green piles previously driven.

Mr. Desmond. THOMAS C. DESMOND, ESQ. (by letter.)—The conclusion of Mr. Cole's paper contains a comparison of the relative supporting power of tapered piles and piles of uniform diameter. The writer respectfully takes exception to this comparison for two reasons: First, Mr. Cole does not compare a pile of uniform diameter with a tapered pile of equal volume; and second, in his computation of "skin-friction" area, he does not take into account the wedge action of the tapered pile, which is a very important factor in its supporting power.

In a thesis on concrete piles, offered for the degree of Bachelor of Science at the Massachusetts Institute of Technology, in June, 1909, the writer made an application of the Rankine theory of earth pressure to a discussion of the relative supporting power of tapered piles and of piles of uniform diameter. As far as he knows, no computations have ever before been published, wherein the reactions of the surrounding earth, which go to make up the supporting power of a pile, have been computed according to any theory of earth pressure. It is respectfully hoped, therefore, that the following computations, which are based on Rankine's theory of earth pressure, may be of interest to the Society.

Two piles will be considered. The first, shown in Fig. 3, is 30 ft. long, with a diameter of 20 in. at the butt, tapering to 8 in. at the point. These are the dimensions of a standard, 30-ft. Raymond pile. The second pile, shown in Fig. 4, is 30 ft. long, with a uniform diameter of 14.42 in. This diameter is choosen in order that the second pile may contain the same volume of concrete as the first.

Computations have been made for the theoretical ultimate supporting power of the two piles, assuming values of B = the angle of repose of earth, ranging from 0° to 30°. The results are plotted in the curves of Fig. 5, where the abscissas represent the values of B, and the

ordinates the supporting power, in tons. The soil is assumed to be homogeneous from the butt to the point of the pile. Mr. Desmond.

In order that the method of making these computations and the assumptions which underlie them may be clearly understood, the computations for the case of $B = 30°$ will be given in full.

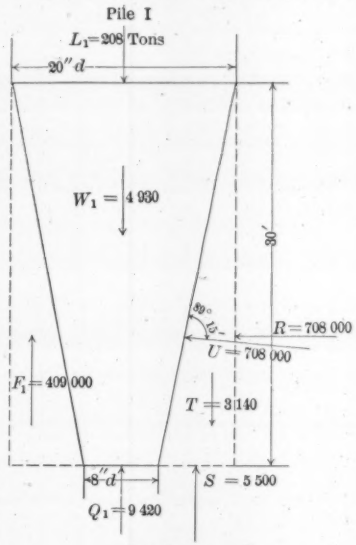


FIG. 3.

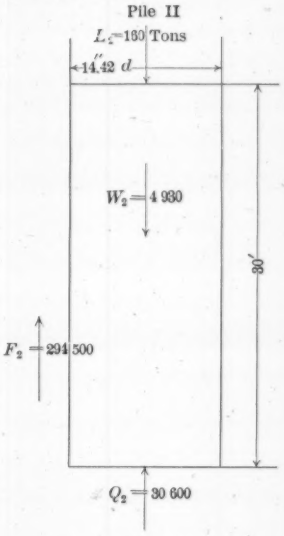


FIG. 4.

Let B = the angle of repose of earth on earth, assumed = 30°,
B_1 = the angle of friction of earth on concrete, assumed = B = 30°,
W = the weight of a cubic foot of earth, assumed = 100 lb.,
G = the weight of a cubic foot of concrete, assumed = 145 lb.,
A_1 = the area of the butt of Pile I = 0.7854 × 400 = 314.16 sq. in.,
A_2 = the area of the point of Pile I = 0.7854 × 64 = 50.27 sq. in.

Then the volume of Pile I

$$= 120\,(314.16 + 50.27 + \sqrt{314.16 \times 50.27})$$
$$= 120\,(364.43 + \sqrt{15\,800})$$
$$= 120\,(364.43 + 125.7)$$
$$= 120 \times 490.1 = 58\,800 \text{ cu. in.}$$

Therefore, in order that Pile II may have the same volume as Pile I, the cross-sectional area of Pile II must be $= \frac{490.1}{3} = 163.4$ sq. in.

Mr. Desmond.

Let d_2 = the diameter of Pile II,

then $d^2_2 = \frac{163.4}{0.7854} = 208.$

Hence, $d_2 = 14.42$ in.

Weight of Pile I = weight of Pile II = $\frac{58\,300 \times 145}{1\,728} = 4\,930$ lb.

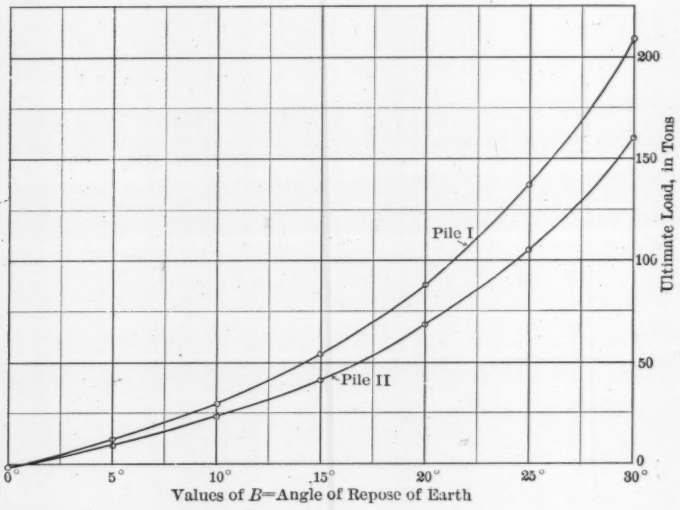


FIG. 5.

Assuming the surface of the ground to be a horizontal plane, and calling P = maximum intensity of pressure, in pounds per square foot, possible on a horizontal plane at a depth, d, below the surface, it is shown in the demonstrations of the Rankine theory of earth pressure that

$$P = w\,d\,\left(\frac{1 + \sin.\ B}{1 - \sin.\ B}\right)^2$$

That is, in this case, when $B = 30°$

$$P = 100 \times 30\,\left(\frac{1.5}{0.5}\right)^2 = 37\,000 \text{ lb. per sq. ft.}$$

Therefore, the supporting force, Q, acting on the point of Pile I = $27\,000 \times \frac{50.27}{144} = 9\,420$ lb. Also, Q_2 acting on the point of Pile II = $27\,000 \times \frac{163.4}{144} = 30\,600$ lb.

Mr. Desmond.

Again, it is shown in the demonstrations of the Rankine theory of earth pressure that, when the surface of the ground is a plane, the pressure on any vertical plane is parallel to the surface, and the intensity of pressure on any plane varies directly with the depth. Also, when the surface of the ground is horizontal, letting q = maximum intensity of pressure possible on a vertical plane, in pounds per square foot, then $q = w\ d\ \frac{1 + \text{sin. } B}{1 - \text{sin. } B}$

That is, for this case, when $B = 30°$,

$$q = 100 \times 30 \times \frac{1.5}{0.5} = 9\,000 \text{ lb. per sq. ft.}$$

Considering Pile II, since the intensity of pressure is zero at the butt of the pile and 9 000 lb. per sq. ft. at the point, and the intensity varies uniformly from butt to point, then the total perpendicular pressure on the lateral surface of the pile

$$= \frac{4\,500 \times 3.1416 \times 14.42}{144} \times 360 = 510\,000 \text{ lb.}$$

Hence, F_2 = the supporting power of Pile II due to friction

$$= 510\,000 \times \text{tan. } B_1 = 510\,000 \times 0.5774 = 294\,500 \text{ lb.}$$

Therefore, the ultimate supporting power of Pile II

$$= 294\,500 + 30\,600 - 4\,930 = 320\,000 \text{ lb.} = 160 \text{ tons.}$$

Let an imaginary cylinder be considered, the projection of which on a horizontal plane is the butt of Pile I, and the outlines of which are indicated by the dotted lines in Fig. 3. Then the total pressure on the lateral surface of Pile I is the resultant of three forces, namely (1), the total pressure on the lateral surface of the imaginary cylinder, acting horizontally = R; (2), the total pressure on the area of the base of the imaginary cylinder less the area of the point of the pile, acting vertically upward = S; and (3), the weight of a volume of earth equal to the volume of the cylinder less the volume of the pile, acting vertically downward = T.

The value of $R = 4\,500 \times \frac{3.1416 \times 20 \times 360}{144} = 708\,000$ lb.

Also, assuming that the intensity of pressure on that part of the base of the imaginary cylinder which is not included in the area of the point of the pile is merely the ordinary pressure due to the depth below the surface of the ground = $w\ d$, according to the Rankine theory, then

$$S = 100 \times 30 \times \frac{(314.16 - 50.27)}{144}$$

$$S = 3\,000 \times \frac{263.89}{144} = 5\,500 \text{ lb.}$$

$$\text{Also, } T = \frac{(360 \times 314.16 - 58\,800)}{1\,728} \times 100$$

$$T = \frac{54\,200}{1\,728} \times 100 = 3\,140 \text{ lb.}$$

Mr. Desmond. Hence the vertical component of the total pressure on the lateral surface of Pile I $= S - T = 2\,360$ lb., and the horizontal component $= R = 708\,000$ lb. Within the limits of slide-rule accuracy, the resultant total pressure on the lateral surface of the pile is therefore also equal to $R = 708\,000$ lb., and the angle which its direction makes with a horizontal plane

$$= \tan.^{-1} = \frac{2\,360}{708\,000} = 0.00334 = 0° 12'.$$

But the lateral surface of the pile is at an angle with a vertical plane $= \tan.^{-1} = \frac{6}{360} = 0.01667 = 0° 57'$.

Hence $U =$ the resultant total pressure, makes an angle

$$= 90° - 0° 57' + 0° 12' = 89° 15'$$

with the lateral surface of the pile, as shown in Fig. 3.

The component of this pressure normal to the surface of the pile

$= 708\,000 \times \text{sin. } 89° 15' = 708\,000 \times 0.9991.$

$= 708\,000$ lb., approximately.

Hence the total frictional resistance of the pile

$= 708\,000 \times \tan. B_1 = 708\,000 \times 0.57735 = 409\,000$ lb.

The vertical component of this frictional resistance $= F_1$

$= 409\,000 \times \text{cos. } 0° 57' = 409\,000 \times 0.99986 = 409\,000$ lb., approximately.

Hence the total supporting power of Pile I

$= 409\,000 + 5\,500 - 3\,140 + 9\,420 - 4\,930$

$= 416\,000$ lb. $= 208$ tons.

The results of the computations, which are plotted on Fig. 5, indicate the superiority of tapered piles as regards theoretical supporting power. It must be carefully noted, however, that these computations are for soils which are homogeneous from the butt to the point of the pile. In the case, so often met in practice, where the piles are driven through soil with a low angle of repose at the butt of the pile and a high angle of repose at the point, that is, where the piles act practically as columns unrelieved of their load until they reach a solid substratum, then, obviously, piles of uniform or increasing diameter would be preferable.

Mr. Coombs. R. D. Coombs, M. Am. Soc. C. E. (by letter).—For years timber piles have been the subject of rather voluminous reports and discussions before various engineering societies and in the technical press, while concrete piles are of comparatively recent development, and accurate data in regard to them are difficult to obtain. Pile-driving is not scientifically determinate, as the conditions governing each case may be different and, in addition, are not usually known.

It would appear to be entirely probable that a type of concrete pile may produce satisfactory results in one instance and prove inadequate

under different soil conditions. In other words, the experience of those who have been engaged in the construction and driving of concrete piles is as yet the sole source of knowledge in regard thereto. Under these circumstances, further information is requested on the following points. Mr. Coombs.

It has been stated by the author—and others—that "cast and driven" piles are not injured by hard driving. Has this fact been sufficiently determined? The writer is aware that some concrete piles have been driven, pulled, and re-driven, and are said to have sustained no injury. In view of the usual complacency with which timber piles are overdriven and the occasional results therefrom, it would seem that, in the absence of tests, no general statement would necessarily cover all types of concrete piles.

If the common practice of using a working load of 25 tons per concrete pile is based on test loads of from 45 to 60 tons (without settlement), can this be considered as being directly comparable to 15 tons per timber pile, when the bearing power of the latter is based on the *Engineering News* formula, which has a theoretical factor of safety of 6?

Referring to the relative dates of completion of the foundations of the warehouse in Pittsburg, given by the author, the comparison may be unduly favorable to the concrete pile method, as the time limit of the ordinary construction is given as the time "set," and it is difficult to understand the reason for a difference in time of 15 months.

Pile *A* is said to resist deformation from earth pressure because the concrete is heavier than the surrounding soil. If the subsoil carries considerable water or is of a very absorbent nature, the cement may be washed out or the water needed in setting may be absorbed by the soil, and, while the theory of earth pressures is not entirely settled, it is possible that a lighter soil may transmit sufficient pressure to distort green concrete.

The writer is perhaps unfortunate, as, in the majority of instances, the piles used in his experience have been called on, at least theoretically, to resist lateral movement or bending. Such piles must necessarily be reinforced, and, in view of the uncertainty of subterranean conditions, either present or future, the addition of reinforcement is well worth careful consideration.

In conclusion, it is to be hoped that the author will give further discussion of the relative merits of straight and tapered piles.

Homer A. Reid, Assoc. M. Am. Soc. C. E.—The old ideas with regard to the bearing capacity of piles may apply to a tapered pile as well as to one which is not tapered. The bearing capacity of a pile foundation depends on increasing the resisting capacity of the soil either by compacting it or by surface friction of the pile and the bearing of the point where it penetrates a firm stratum. This fric- Mr. Reid.

Mr. Reid. tional resistance is considerable. Patton states that piles driven in the semi-fluid swamps of the South were able to carry quite heavy loads, although at the time of driving they sank 15 or 20 ft. under the weight of the hammer alone, and only a few light blows were needed to drive them their full length; a day or two later it was found that additional penetration could not be secured without several heavy blows of the hammer.

If the speaker is not mistaken, this type of pile has a smooth shell which would have a very low adhesive value; hence its bearing capacity would depend on its compacting the soil, and, to a greater or less extent, on the bearing capacity of the soil.

The depth to which a pile may be driven has something to do with its permanency in a foundation. If a comparatively short tapered pile is highly loaded, and, due to certain conditions which may arise in the future, the soil is disturbed in some way, such a pile may not prove successful, and it would seem that another type, perhaps one having greater length, would be of more value as a permanent foundation.

Piles classified by the author as Type *A* have been used quite successfully in many parts of the United States, but in some cases they have been failures.

In New York City, some two or three years ago, an attempt was made to use this pile in the construction of the foundations of the Municipal Lodging House at 25th Street and the East River. Owing to a change in plan—the addition of a deep sub-cellar, or something of that nature—the piles were dug up. Trouble had been experienced with them before the change of plan, and when they were dug up it was found that they had separated into sections about 5 ft. in length. The speaker was not able to obtain photographs, but on examining the sections closely it was seen that at the time of driving the piles the fluid alluvial soil had penetrated between individual batches of the concrete, making distinct cleavage planes, separating the piles into sections, which, when hardened, came out in distinct cylinders about 5 ft. in length. One might speculate in regard to the bearing capacity of piles of this nature.

In the use of piles of this type, considerable difficulty was also experienced in another large foundation operation within the bounds of Greater New York. In this case the difficulty appears to have been due, in a measure, to the use of a poor and improper aggregate and to haste in driving and withdrawing the shell; the character of the subsoil also, and possibly the character of the water in the subsoil, had something to do with the failure of the piles. The speaker knows of a concrete foundation operation in the immediate vicinity where the cement failed to set up, which failure was shown by chemical analysis to be due to the character of the water in the soil.

In making Type *B* piles, unless the work is done under an exacting

inspector, it is not customary to tamp the concrete, and it would seem doubtful whether a compact concrete would result. Mr. Reid.

The speaker has been informed that in one instance, when such a pile was dug up, it was found that the concrete had bridged, leaving a space of several inches between the upper and lower sections.

Of course, in firmer soils, a different condition would arise, and there would be no danger of a penetration between the successive batches of concrete. The speaker thinks that the adhesive qualities of the soil resulting therefrom are more to be depended on than its additional firmness due to the wedging action of a tapered pile, where a concrete pile can be used, as is done in the later or improved method of making Type *A* piles, that is, a pile in which a tube is drawn. He has been informed that in some recent methods the concrete is driven or held in place by a heavy weight as the shell is drawn, thereby forcing it into the soil and making a foundation which would have a maximum adhesion, varying with the compactness of the soil.

Another method, which has been in use in Europe and in one operation going on at the present time in the United States, is that in which no shell at all is used, but a weight of the shape and form of a plumb-bob, weighing 2 or 3 tons, is lifted into the air and dropped several feet, perforating and compacting the soil. By dropping it successively in the same spot, the soil is gradually forced to one side and compacted. This operation is continued until the point penetrates a firm stratum, at which time a rammer having a blunter point, similar to a half-section of an acorn, replaces the cone-shaped point, and compresses the soil still more. Large stones, gravel, and concrete are then placed in the hole and repeatedly rammed with the second punch. This forces the concrete sideways into the soil and produces a large pillar which, when hardened, is capable of sustaining very heavy loads. The operation is completed with a rammer having a flat-bottomed surface, which compacts the concrete firmly and evenly.

When a semi-fluid soil or quicksand is met, the use of balls of clay forced into the soil, in a manner not unlike that used in tunnel and open-caisson work, enables the operation to be carried forward. The simplicity and cheapness of this operation, the thorough assurance that a good, compact concrete will be obtained, together with the certainty of a stable and massive concrete foundation, will make this method popular with the engineer and the owner.

In regard to the use of steel piles, the author's criticism, in which he states that the life of the steel in the exterior shell is somewhat doubtful on account of oxidation by the ground-water, the corrosive action of gas in the soil, or stray electric currents, is hardly borne out by facts. No oxidation can take place if the water is not continuously changed, as it is the free water which causes oxidation. As to the corrosive action of gas in the soil, it would seem that where piles are

Mr. Reid. used, little or no gas would be met. As regards the action of electrolysis due to possible currents in the soil, it is hardly probable that a continuous flow of current can take place, as the tops of the piles are encased in concrete which, in itself, is one of the best insulating materials known.

As to the preservation of metal in the ground, the speaker is informed by Mr. Thomas Rowe, Architect, that he had removed from the ground, in Lower New York, cast-iron pipes which had been in place for approximately seventy years, and it was found that their outer surfaces were still in good condition, practically no rusting having taken place.

Charles H. Nichols, M. Am. Soc. C. E., while constructing an amusement park, at Dreamland, Coney Island, in 1903, removed three old cast-iron piles which constituted a portion of the old iron pier. It was found that between the ground line and a point 10 ft. above low water evidence of corrosion was very distinct, but below the ground line it was very slight, and those portions which were below the ground line were absolutely unchanged. This was also true of the cast-iron shoes at the point of the pile.

In an informal discussion before this Society in 1907, John F. O'Rourke, M. Am. Soc. C. E., made the following statement:*

"Iron and steel used in foundations, apart from conditions where electrolysis may occur, last indefinitely when in direct or indirect contact with water, provided the water remains unchanged. The reason for this is obvious. Water attacks iron or steel on account of the oxygen it contains, and, if this is a proportionately small quantity, the amount of oxygen contained in wet concrete or ground is negligible, and, having once been exhausted, the metal remains unharmed and protected.

"The writer has seen many cases where immersion in standing water has been a matter of years, and in every case the effect upon the metal has been no greater than if it had stood for the same length of time in linseed oil. In one case bolts on the inside of cast-iron cylinders, filled with concrete, were exposed to the salt water in the Harlem River for more than 30 years, and when removed were found to be without rust. In another case a pipe was immersed for ten years in an Artesian well, the water in which had not been pumped for 10 or 15 years, and no corrosion of this inside pipe had taken place, the scale was still as fresh as when the pipe was new, and the tool marks of the pipe-coupling apparatus were still perfectly fresh."

Just a word in regard to safe allowable loads on piles: A practice has sprung up in New York City within the past few years of assuming a load based on a pressure of 350 lb. per sq. in. on the upper end of the pile, irrespective of the length or character of the stratum into which the pile is driven. The speaker is unprepared to say who is responsible, but, from an engineering point of view, it appears to be

* *Proceedings*, Am. Soc. C. E., August, 1907.

FIG. 1.—VIEW OF EXCAVATED PILE.

FIG. 2.—VIEW OF "FOOT" OF PILE.

decidedly illogical. Loads varying from 30 to 50 tons have been asked for by those interested in the use of Type *B* piles, the assumption being that the top end of the pile was either 16 or 20 in. in diameter. In some cases when the piles were driven it was found impossible to force the punch into the ground more than from 8 to 12 ft. On account of its tapering shape, the top of the pile at the point of cut-off would not be more than 12 or 15 in. in diameter. No additional piles were provided, and the use of such short piles with high loads can under no circumstance be considered as sane or safe engineering practice. Mr. Reid.

It would appear to the speaker that the only safe and equitable basis of letting a contract for pile work would be to assume a safe bearing load, say of from 1 to 1½ tons per ft. of penetration, together with a unit price per foot of penetration starting with a depth of, say, not less than 10 ft. In this case nothing would be gained by the contractor in not driving the piles to a safe depth. It should be noted that, in regard to the use of concrete piles, conditions in New York City are entirely different from those elsewhere, owing to the confined spaces within which the work has to be done. It is well to emphasize the fact that in New York City if a big pile is driven alongside an old foundation wall and forced down until the top—which may be 20 in. in diameter—is level with the surface, with a penetration of 18 or 20 ft., the earth around for several feet is disturbed. This necessitates needling the walls and holding them in place, and even when this is done, there is considerable difficulty in wedging up the walls and letting them down carefully without destroying the old building.

HUNLEY ABBOTT, JUN. AM. SOC. C. E. (by letter).—Mr. Cole's paper is the most complete description of modern American methods of driving concrete piles that the writer has seen. In his discussion of the various types of piles he combines justice with good engineering judgment, which is by no means an easy thing to do. Mr. Abbott.

The writer has recently devised a new method of concrete pile construction, a description of which may serve to round out Mr. Cole's paper.

It has long been recognized that great additional strength and stability may be obtained by having an enlarged foot or base at the lower end of a pile. More than a hundred years ago screw- and disk-piles were used in England and in the United States. Rankine mentions some cast-iron piles, from 20 to 45 ft. long, with screws 3 ft. 6 in. in diameter, which carried a working load of 67 tons per pile without settlement. Mr. Cole mentions some cast piles with bulbous ends, which were used at Atlantic City; also, some cast-in-place piles which were made by chambering out a cavity with a toggled cutting tool, at the lower end of the pile. Hitherto, although it has been generally recognized that an enlarged foot is a desirable thing, the trouble has been to get that foot down. The screw-pile necessitates a very slow

Mr. Abbott. and laborious method of placing, the disk-pile can be used in only one kind of soil, and the difficulties of chambering out a cavity, 20 or 30 ft. below the surface, through a small pipe, are obvious.

In the writer's method an enlarged foot is formed at the lower end of a cast-in-place pile in the following manner: The apparatus necessary to form the pile consists of a casing and a core. The casing is a steel pipe, 16 in. in diameter and $\frac{3}{8}$ in. thick, and the core is a smaller and longer pipe, with a cast-steel point and an enlarged cast-steel head. The core fits inside the casing, its enlarged head engaging the top of the casing and its lower end projecting some 4 or 5 ft. below the lower

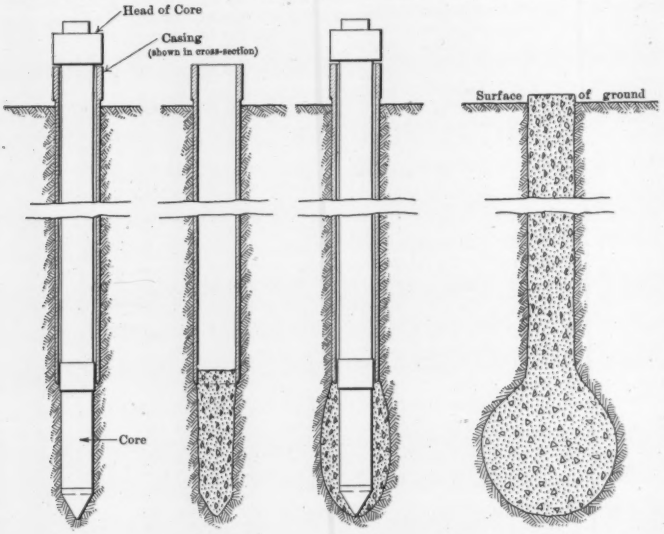


FIG. 6. FIG. 7. FIG. 8. FIG. 9.

end. In the head of the core there is an oak driving block which receives the blows of the hammer. The core is fitted into the casing and both are driven into the ground to the desired depth, as indicated in Fig. 6.

The core is then pulled out, and a charge of concrete is dropped to the bottom of the casing, as in Fig. 7. The core (or rammer, as it now becomes) is lowered into the casing and driven through this charge of concrete, pushing it to either side, as shown in Fig. 8. This operation of placing charges of concrete and ramming them is continued until a large bulb or foot of the desired size is formed, as shown in Fig. 9. If the soil just below the foot is somewhat harder than that

FIG. 1.—CORE HANGING IN LEADS OF PILE-DRIVER.

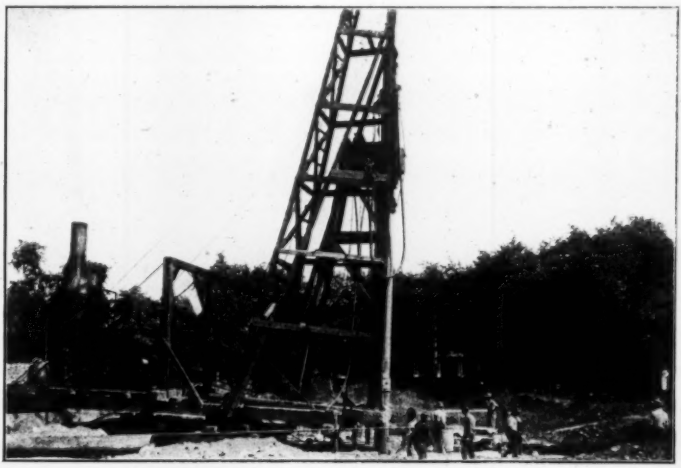


FIG. 2.—PULLING THE CASING.

above it, the foot may flatten out on the bottom to some extent. After the foot has been formed, the casing is filled to the top with wet concrete and then pulled out, leaving a concrete column 16 in. in diameter, resting on a spread base or pedestal. This column may be reinforced with steel rods if greater strength is desired in that part of the pile. Mr. Abbott.

It is evident that this pile will carry a considerably greater load under given conditions than a straight pile without the foot. It has all the frictional area along the stem that the straight pile has, and also some around the horizontal circumference of the foot. In addition to this friction, it has the bearing value of the projected area of the foot on the firm soil at the lower end of the pile. This soil has been compressed and compacted by the ramming of the concrete during the formation of the foot, and, therefore, is capable of bearing a greater load than it was before the driving of the pile. Experiments have been made with this pile in a number of different kinds of soil, and it has been found that the shape taken by the foot is, in all cases, roughly spherical, as shown in Fig. 9.

Some loading tests in the yards of the Lackawanna Steel Company, at Buffalo, showed that, in the soft muck along the lake front, the bearing power of a concrete pile with an enlarged foot was two and one-half times that of a pile of the same length without the foot. Other tests, made in St. Louis, showed that a pile of this new type, 24 ft. long, would carry a greater load than a straight pile of nearly twice that length. The soil in this case was a cinder and trash fill on yellow mud, and the test load was 40 tons. This pile was dug up, and two views of it are shown on Plate XXXVII. The shaft of this pile was 16 in. in diameter and the foot had a horizontal diameter of 4 ft. 3 in.

Some of these piles have recently been driven for the foundation of the new post-office building at Paris, Ill. They averaged 19 ft. in length (measured from top of head to bottom of foot), and penetrated through a stratum of from 2 to 4 ft. of quicksand. Plate XXXVIII shows the pile-driver at work on this job. In Fig. 1, Plate XXXVIII, the core is seen hanging in the leads, while a charge of concrete is being thrown into the casing. Fig. 2, Plate XXXVIII, shows the pulling of the casing, which is the last step in the formation of the pile.

Howard J. Cole, M. Am. Soc. C. E. (by letter).—The twin jetting device on the Columbia River, as described by Mr. Tower, is similar to that used in the East, with some variations in sizes. On some timber pile work in beach sand in the fall of 1908, where a uniform pile length of 20 ft. was adopted, the jet pipes were 22 ft. long (2 in. in diameter contracted to 1 in. at the outlet), connected at the upper end by a 4-in. elbow and a nipple having a T-connection to which was attached the rubber hose leading to the force main. The whole apparatus was suspended in position by a tripod, from 20 to 25 ft. high, composed of Mr. Cole.

Mr. Cole. 4 by 4-in. timbers, which could be shifted readily by the jetting crew. The jets were forced into the sand deep enough to make a hole in which to stand the pile, and were then withdrawn. The pile was then placed in the jetted hole, and forced to the proper position. When the pile reached its bearing, the flow was shut off, and the jetting crew stood on the pile to overcome its buoyancy until the sand had set against it. In jetting concrete piles, their weight is a great help in following down the jet. The timber piles on this job were jetted 18 in. apart, from center to center, without disturbing the piles previously placed.

In New York City there are many instances where steel-cased piles, referred to by Mr. Mensch, are used in shoring operations. They are placed in position in contracted quarters, where there is but little head-room, and, consequently, are put down in sections, the steel casing (being a means for constructing the concrete pile) is forced by jacks, or a hydraulic hammer, to the bearing stratum, and is then cleaned out and filled with concrete. Reinforced moulded concrete piles, under these conditions, could not be used unless there was a stock of seasoned piles of various lengths from which to draw. The writer understands that one of the moulded-pile companies is now contemplating the immediate construction of a stock yard in which will be kept a large supply of moulded piles of standard lengths, from which rush orders can be filled.

The owner who has had to put in a concrete pile foundation has usually taken a made-in-place type, because he could commence work and drive piles immediately on the signing of the contract, whereas, with the moulded or cast types, at least 3 weeks' time is lost waiting for the piles to season.

The Compressol method, mentioned by Mr. Mensch, has not been used at all in the United States, as far as the writer has been able to find out, and only to a limited extent abroad.

The timber piles used in the traveler supports, to which Mr. Higgins refers, were selected sticks, carefully driven by experienced drivers, and, although intended only for temporary use, great care was taken to secure a good job. Notwithstanding all these safeguards, many of the piles were found to be injured.

The word "comparable" (near the foot of page 469), is used advisedly, as one would hardly expect to find many timber piles of the same dimensions as the standard concrete pile, say 16 in. in diameter, throughout their entire length, or tapered with a 22-in. butt and an 8-in. top. In general, timber piles are less than 12 in. at the butt, unless they are specially selected and an extra price is paid for them, so that a timber pile of comparable size would be one of ordinary dimensions, as used in common practice.

Mr. Higgins takes exception to the relative cost of concrete and timber piles at Annapolis, Md., as given in the paper, and works up a

possible comparative cost based on an assumed hypothesis. According to the inspector in charge of that work, it was estimated that, judging from the timber piles driven for an adjoining building, piles 40 ft. long would be required, would cost 20 cents per foot, and would be driven 30 ft. below mean low water at an additional cost of 5 cents per foot, or a total of $9.50 per pile. The length of the longest concrete pile driven was 30 ft., and the average of all was about 16 ft., which makes the price, $20 per pile, a reasonable figure. Mr. Cole.

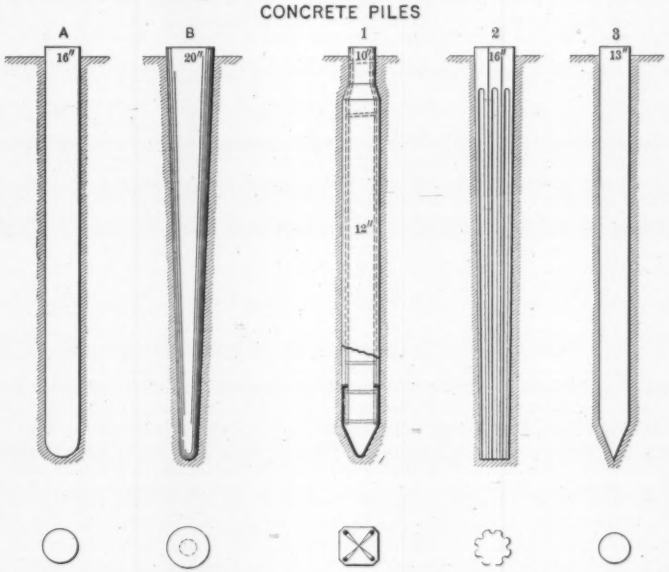


FIG. 10.

There are several modifications of the composite timber and concrete pile on the market, and also various combinations of "made-in-place" tops; and cast or moulded tops have been suggested. The main difficulty is in making a thoroughly satisfactory connection between the timber pile and its concrete top. The general method is to use dowels, or exterior sleeves, or both. In many instances, where the alluvial soil is of great depth and the piles cannot be driven to a firm stratum, the load must be supported entirely by friction, and the lighter composite pile will be much better adapted to the conditions than the heavier concrete pile, which would increase the load very considerably without any compensating benefit.

Mr. Cole. Mr. Upson is entitled to credit for the work he has done in tabulating some of the records of his various contracts, with the idea of obtaining some empirical formula for the carrying power of the Type "B" pile. His chart, Fig. 2, shows that the pile marked *B*, with the decreased taper, was driven to a greater penetration than the pile marked *A*, having the maximum taper, which shows that it is possible to secure a deeper penetration with a straight pile than with a tapered one, and thus reach the lower lying stratum with the foundation. The heavily tapered pile acts as a wedge, and, as the weight of the core and the weight of the hammer are practically the same, after a certain depth has been reached, the effect of the hammer is overbalanced by the soil compression and the friction of the wedge, making it impossible to drive any farther, even though a good bearing stratum may be but a few feet away. As has been stated before, the tapered pile depends for its support largely on the soil compression of the upper layers of the ground, and if anything should occur to remove the surface stratum, there would be a possibility of a settlement of the structure. That Mr. Upson's company is now prepared to put in straight piles, after driving tapered piles for so many years, is an admission of the merit of the straight pile.

Mr. Desmond, after a mathematical demonstration in which he proves the superiority of the tapered over the straight pile, says:

"It must be carefully noted, however, that these computations are for soils which are homogeneous from the butt to the point of the pile. In the case, so often met in practice, where the piles are driven through soil with a low angle of repose at the butt of the pile and a high angle of repose at the point, that is, where the piles act practically as columns unrelieved of their load until they reach a solid substratum, then, obviously, piles of uniform or increasing diameter would be preferable."

In reference to Mr. Coombs' inquiry about the time saved on the Pittsburg warehouse foundation, each contractor, when bidding on the work, inserted in his proposal the length of time that he would require to complete the foundation by his method. The shortest time given in the proposals of the open-pier-method contractors was 18 months, but the foundation was completed by the use of concrete piles in 76 days, making a saving of 15 months by the adoption of a foundation of this type. Mr. Coombs appears to doubt the value of a directly-applied test loading on concrete piles of double the load that the pile is to carry. When concrete piles were placed on the market, there was only one way of convincing the owner that they would support his building, and that was by making an actual demonstration before his eyes by loading the pile to more than it was to carry. This method was more conclusive to him than any formula would have been, even though that formula had a theoretical factor of safety of 6.

Mr. Cole. Mr. Reid mentions some of the failures of concrete piles. There have been failures in all the types, and it is owing to experience gained in this way that the better concrete pile of to-day has been evolved. Many of the early failures were due to improper methods of installation, or imperfect ingredients in the concrete, or to lack of information regarding subsurface conditions. In one case, recalled by the writer, considerable time and money were lost by driving the piles instead of jetting them; this very nearly made a failure of a promising proposition.

Mr. Reid states that water pipes have been removed from streets in the lower part of New York City after being buried for 70 years, and were found to be in good condition, but he does not state that subsurface conditions have changed very considerably in the past twenty years, and that the gas-impregnated soils and electric currents now cause trouble to all metal buried beneath the surface. The water departments of all large towns and cities have suffered losses in their mains by electrolysis. Conditions in New York and other cities, where electricity is used so extensively, have changed materially in the last decade, and it is not safe to compare the effect of the soil on buried metals in ante-bellum days with that of the present.

The made-in-place pile with an enlarged foot, described by Mr. Abbott, is an adaptation of the Type "A" pile, and, under favorable subsoil conditions, should make a good foundation; the weight of the imposed load is distributed over a larger area, thus giving a smaller unit load per square foot than is possible with the ordinary types of piles.

This paper was presented for the purpose of bringing foundation work of this type more generally to the attention of engineers, and to promote knowledge through its discussion.

MEMOIRS OF DECEASED MEMBERS.

GEORGE ADGATE, M. Am. Soc. C. E.*

DIED JANUARY 23D, 1909.

George Adgate was born on September 28th, 1845, at Keeseville, New York, and was the eldest of a large family of children. His ancestors emigrated from England to Massachusetts and Connecticut during the earliest Colonial times. Through a marriage between the son of the first Adgate who came to this country, and a granddaughter of Elder William Brewster, the Adgates were placed in the line of descent from the Pilgrims.

The family has been useful and influential in public affairs throughout its history. Mr. Adgate's great-grandfather, Judge Matthew Adgate, was a member of the New York Legislature and of the convention which formulated the first constitution of the State of New York. His grandfather and father were also members of the New York Legislature.

After his early education at Keeseville Academy, one of the best of the old-time schools, Mr. Adgate spent two years keeping a store in the oil regions of Pennsylvania. Later he became connected in business with his father-in-law, Mr. Whitney, a contractor on Lake Champlain. The work consisted principally of dock-building, much heavy construction of that kind being necessary in those days, before any railroads had been built on the shores of the lake. The fifteen years he spent in struggling with the difficulties of subaqueous work, often under severe climatic conditions, were instrumental in developing his judgment and resourcefulness.

In 1882 Mr. Adgate was engaged by William Sooysmith and Son as Manager of bridge foundation construction, most of which involved the use of pneumatic caissons. He was engaged with this firm and its successors throughout the remainder of his busy life. Here his experience and sound judgment were often tested while building foundations in large rivers where varying conditions as of floods and ice gorges had to be anticipated and provided for in advance. In this warfare with the elements it can be truthfully said that he never lost a battle.

He was in active and responsible charge of the building of the foundations and piers of the following bridges:

Terre Haute, Ind. "Big Four" Bridge over the Wabash River.
Van Buren, Ark. Bridge over the Arkansas River.

* Memoir prepared by E. L. Abbott and Daniel E. Moran, Members, Am. Soc. C. E., and Edwin S. Jarrett, Assoc. M. Am. Soc. C. E.

Needles, Cal. Bridge over the Colorado River.
Cuivre Junction, Mo. Bridge over the Cuivre River.
Boone, Iowa. Chicago and North-Western Bridge over the Des Moines River.
Harvey, Iowa. Chicago, Rock Island and Pacific Bridge over the Des Moines River.
Kansas City. Chicago, Rock Island and Pacific Bridge over the Kaw River.
Clinton, Iowa. Chicago and North-Western Bridge over the Mississippi River.
Ottawa, Can. Interprovincial Bridge over the Ottawa River.
Glasgow, Mo. Bridge over the Missouri River.
East Omaha, Nebr. Bridge over the Missouri River.
Plattsmouth, Nebr. Chicago, Burlington and Quincy Bridge over the Missouri River.
Sibley, Mo. Bridge over the Missouri River.
Sioux City, Iowa. (Pacific Short Line) Bridge over the Missouri River.
Arthur City, Tex. Bridge over the Red River.
Gainesville, Tex. Bridge over the Red River.
Fort Worth, Tex. Gulf, Colorado and Santa Fé Bridge over the Trinity River.
Marietta, Ohio. Bridge over the Ohio River.

He was also in charge of many smaller bridges.

Mr. Adgate was associated in the construction of the foundations of the Baltimore and Ohio Bridge across the Susquehanna River, at Havre de Grace; he also supervised the sinking of several deep shafts and the building of numerous docks, dams, intake piers, lighthouses, and other foundation work. At the time of his death, January 23d, 1909, he was Western Manager for The Foundation Company of New York City.

Mr. Adgate inherited a wholesome and rugged constitution, which enabled him to contend successfully with the many adverse conditions and surroundings incident to his work in the difficult kinds of construction on which he was continuously engaged.

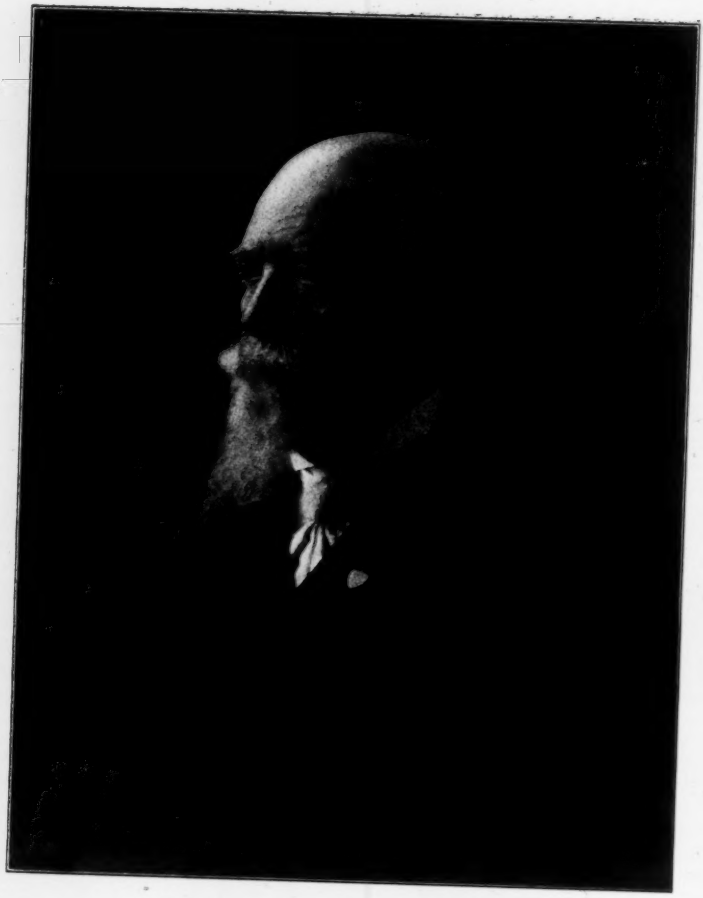
His organizing ability was wonderful, and his keen foresight, calm judgment, and iron determination were chiefly instrumental in the success of the many works for which he was responsible.

Personally, Mr. Adgate was kind, just, and lovable. He could direct the operations of others so as to obtain their best efforts and loyal support, and the engineers and associates with whom he worked, had, to the last, the highest regard for his ability and integrity. He had a fund of quiet humor, which came to the aid of his associates and helpers in the most trying situations and lightened their load.

He was always ready to help young men after they had shown their worth, but believed in trying them out in a rough but kindly way. He was genial and bluff, but on questions of principle stood like a rock against persuasion or threat.

He was married on September 19th, 1866, to Miss Martha Whitney, who survives him, together with a daughter, Mrs. J. R. Dean, of New Britain, Connecticut, and a son, Frederick W. Adgate, Assoc. M. Am. Soc. C. E., of Chicago.

Mr. Adgate was elected a Member of the American Society of Civil Engineers on April 1st, 1896.



WILLIAM PRICE CRAIGHILL

WILLIAM PRICE CRAIGHILL, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1909.

A long and active life spent in the service of the country builds its own enduring monuments. Little remains to his friends other than to pen a few inscriptions, which, though inadequate, may serve to remind the passer-by more clearly of the man whose work is there in testimony of the duty done and the good accomplished. An engineer, eminent in his profession, a Past-President of the American Society of Civil Engineers, and for a time the head of his Corps in the Army, must necessarily have such monuments widely scattered over our land. It is so with General Craighill. The records of his work are found in our improved rivers and harbors, in our fortifications, and also in the lives of the younger men of the profession who were associated with him and whom it was his delight to help.

William Price Craighill was born in Charlestown, West Virginia (then Virginia), on July 1st, 1833, the eldest son of William Nathaniel and Mary Elizabeth (Brown) Craighill. He was of mixed English and Scotch blood, and his ancestors bore a prominent part in the early history of Virginia. The earliest of the American Craighills, William, is believed to have been born near Carlisle, England. His son, Nathaniel, served in Captain William Washington's company of Minute Men, in 1774-75, and later was in the Continental service. The first William Price Craighill, son of Nathaniel, was born at Craighill Point, Westmoreland County, Virginia, served as a volunteer in the War of 1812, and was a member of the Legislature of Virginia about 1817. General Craighill, his grandson, was named after him. Among other ancestors can be named the Littles, one of whom was a friend and neighbor of General Washington at Mount Vernon, and one of his pall-bearers; the Browns, who claim descent from one of the founders of Jamestown; and the Rutherfords, of an ancient Scottish family, one of whom, Robert, the maternal great-grandfather of General Craighill, was also an associate and friend of Washington, as a surveyor and fellow-officer of the early Indian wars, and a participator in the political agitations in Virginia immediately preceding the Revolution. He held many places of trust in Virginia, and was the first Member of Congress elected from the west of the Blue Ridge, having represented Berkeley County from 1793 to 1797.

Among the founders of the Charlestown Academy, in 1795, were three of the grandfathers of General Craighill, Little, Craighill, and Rutherford. Here the General received his early schooling, following his grandfather and father, and here his sons succeeded him. General

* Memoir prepared by the following Committee: General Charles W. Raymond, Colonel William M. Black, Lieutenant-Colonel Gustave J. Fiebeger, Bernard R. Green and John Thomson, Members, Am. Soc. C. E.

Craighill was for years a Trustee of the school, and finally President of the Board. On July 1st, 1849, at the age of sixteen, he became a Cadet at the United States Military Academy, the youngest of his class, the roll of which contained many names destined for fame, and among these may be mentioned McPherson, Schofield, Sheridan, Sill, Sooy-Smith, Terrill, Vincent, Tyler, Sweitzer, and Hood. He was graduated in 1853, second in the class, which McPherson led, and was commissioned Brevet Second Lieutenant, Corps of Engineers. His subsequent advances in rank in the Corps of Engineers of the Army were as follows: March 3d, 1855, Second Lieutenant; July 1st, 1859, First Lieutenant; March 3d, 1863, Captain; November 23d, 1865, Major; January 2d, 1881, Lieutenant-Colonel; January 10th, 1887, Colonel; May 10th, 1895, Brigadier-General and Chief of Engineers. On March 3d, 1865, he was made Brevet Lieutenant-Colonel "for faithful and meritorious services during the war, and particularly for services in the defence of Cumberland Gap, and the ulterior operations of General Morgan's forces." He also received the brevet of Colonel "for gallant and meritorious services during the rebellion," but this he declined.

When the Legislature of Virginia adopted the ordinance of secession, in 1861, Lieutenant Craighill was confronted by the grave question whether he should follow the fortunes of his native State or remain in the service of the Union. On the one hand were all the associations of his youth, the pride of his ancestry, and the certainty of immediate professional advancement; on the other hand, a solemn sense of duty and the sincere belief that the ultimate welfare of the whole country depended on the success of the cause of the Union. After the most anxious and thoughtful consideration, he decided that it was his duty to remain in the service of the Union; but he refused to seek advancement in a war in which he found himself opposed to his own people, and thus voluntarily surrendered the opportunity of obtaining the high honors and personal distinctions which were attained by so many of his associates of similar age, training, and experience in both the Confederate and the Union armies.

From 1853 to 1856, Lieut. Craighill was on duty on the improvement of the Savannah River and in the construction of fortifications at its mouth, on fortification work at Dry Tortugas, and in fortification construction and harbor improvements in Charleston Harbor. He was stationed at Fort Sumter during the yellow fever epidemic which ravaged Charleston in 1854-55, and while there became cognizant of one of the terrible abuses which slavery made possible, which made on his mind an indelible impression of the evils of the system. In 1856, he went to Washington as Assistant to the Chief of Engineers, General Totten, Hon. M. Am. Soc. C. E. In later life, he was fond of recalling the methods of that talented and painstaking official, who, with his own

hands, frequently made working drawings of the details of the beautiful stonework to be found in the fortifications of that date, showing a possibility for the careful individual study of the problems of the office, which must be a cause of envy for General Totten's overworked successors of later days. In 1859, Lieut. Craighill was ordered to the Military Academy as Principal Assistant Professor of Engineering, and remained on that duty until June, 1863. While at the Academy, he served for two years as Treasurer, and for one year was in command of the detachment of Engineer soldiers at West Point. He also compiled "The Army Officer's Pocket Companion," a treatise which proved very useful to the Volunteer Army, and translated Dufour's "Cours de Tactique" and, jointly with Captain Mendell, Jomini's "Précis de l'Art de la Guerre."

In the summer of 1862 he was Chief Engineer of the Division of General George W. Morgan, Army of the Ohio, and was engaged in the defense of Cumberland Gap. In his report on the campaign, dated October 12th, 1862, General Morgan writes:

"Nor can I close this report without calling the attention of the Commanding General to the important services rendered me by Lieut. W. P. Craighill of the Engineer Corps. He is an officer of distinguished merit, and is thoroughly informed on all subjects connected with the art of war. He would make an able Chief of Staff, or fill with high credit any other position to which he may be assigned, and deserves a much higher grade than he now holds."

In the summer of 1863, Captain Craighill was Chief Engineer of the Department of the Monongahela, and was engaged in the construction of the defences of Pittsburg, then threatened by the raid of General John Morgan. Later, he was Assistant Engineer in the construction of the defences of Baltimore, and in 1864, was Chief Engineer of the Middle Department and Eighth Army Corps. In 1864, he again went to the Military Academy, where he served as Assistant Professor of Engineering, Instructor of Practical Military Engineering, in command of the Engineer Detachment, and as Treasurer, serving also on Boards for the defences of San Francisco, for fortifications at Willets Point, and for New York.

In November, 1865, he was sent to the Baltimore District, in charge of the construction of the fortifications and of river and harbor improvements, and, with the exception of the tour of duty from 1866 to 1870 as Assistant to the Chief of Engineers, General A. A. Humphreys, Hon. M. Am. Soc. C. E., he remained on that duty until promoted to be Chief of Engineers in 1895. His fortification work included the construction of works at Baltimore, Washington, and Hampton Roads. The river and harbor works extended from the Susquehanna to the Cape Fear, and as far west as the Great Kanawha, in West Virginia. In Baltimore Harbor, the dredged channel constructed by him bears his name.

The improvement of the Cape Fear River presented a notable engineering problem, and the successful closing of the false mouth across the shifting sands of the coast was a work of great importance. In this work was evolved the log and brush mattress, used extensively later on the South Atlantic Coast. Perhaps the most notable and successful of the works was the canalization of the Great Kanawha River, in which the Chanoine system of movable dams is used. It is believed that the introduction of this system in the United States is mainly due to the late Colonel Merrill, with whom Colonel Craighill visited Europe in 1878, for an inspection of works of that character; but the first Chanoine dams actually constructed and placed in service in the United States were on the Great Kanawha River, under Colonel Craighill.

In addition to these regular duties, Colonel Craighill served on many boards formed for the consideration of projected improvements, among which may be named those for the Columbia, Mississippi, Hudson, Ohio, Delaware, Potomac, Cape Fear, Great Kanawha, James, and other rivers, and the breakwaters for Delaware Bay, Sandy Bay, Massachusetts, and San Pedro, California. For several years he was a member of the Light-House Board and of the Board of Engineers in New York, to which were referred questions relating to fortifications and river and harbor improvements.

In 1884 the system of grouping a number of river and harbor districts into a division was inaugurated. From that date until 1895 Colonel Craighill was the Division Engineer of the Southeast Division. When this division was first formed, it had several of the younger officers of the Corps of Engineers as its district engineers, and was known as "the kindergarten." The Division Engineer's visits were always welcome, and he took a keen interest in the work of the younger men, approving their methods whenever practicable. "For," he said, "I have found that generally there are several ways of accomplishing a given result, and that it is best to follow the plan of the man who is to do the work providing that plan be sensible"—a truth not always appreciated by superiors. At another time, he said to a young officer who was reporting to him for duty, "Mr. ———, I propose to be the laziest man in this district, and do not propose to do anything that I can have done by one of my assistants." It is needless to say that, under such a man, the assistants always did their best.

As Chief of Engineers, it fell to General Craighill to arrange for the prompt construction of the many new works of defence for which Congress had for the first time for years made large appropriations. Under him, a number of changes were made in the methods and policy of the Engineer Department, the most important of which was the inauguration of the practice of having the type plans of the batteries, with the general project for the defense of a harbor, referred to the

local engineer, prior to final adoption, in order to obtain the benefit of his local knowledge and experience. Criticisms of the plans were requested from the various branches of the service interested. The results were immediate and beneficial. Due to his insistence, the development of the disappearing carriage for 12-in. guns was hastened by the Ordnance Department, thus permitting the abolition of the very expensive lift type of battery, which, before that, was the only type for 12-in. guns affording protection while they were being loaded. He also changed the type of the mortar batteries, and had important improvements made in the submarine mine system. Under him, was begun the practice of disseminating important technical information among the officers of the Corps by means of mimeographs.

Incident to his service, General Craighill had to appear frequently before the various committees of Congress, where he made many friends. His succinct and straightforward answers to questions were always received with marked attention. His sound judgment and evident thorough knowledge of the subject at issue caused his opinions to have great weight. An evidence of this is found in the history of the Washington Aqueduct tunnel. This had been abandoned after the discovery of fraudulent work on the part of the contractors, and the tunnel project had become thoroughly discredited in Congress. After an expert examination into its merits, General Craighill presented to Congress a full and convincing report, and, as a result, the necessary appropriations were made and the work was successfully carried to completion. The tunnel to-day forms an important link in the Washington water-supply system.

On February 1st, 1897, General Craighill was retired from active service at his own request. Excepting a limited service as a member of the Board of Consulting Engineers of the Dock Department of the City of New York, his retirement ended his active career as an engineer. He died at Charlestown, West Virginia, on January 18th, 1909. The immediate cause of his death was stated by his physicians to have been "disability following a stroke of apoplexy causing paralysis of the right side."

General Craighill was married on October 14th, 1856, to Mary A. Morsell, daughter of Judge James S. Morsell, of Washington, D. C., and, after her death, was married, on September 22d, 1874, to Rebecca Churchill, daughter of the Reverend Alexander Jones, of Virginia. He had three sons and three daughters. One of the sons, William E. Craighill, was graduated second in the class of 1885, United States Military Academy, and is now a Major of the Corps of Engineers, U. S. Army.

Of civic duties General Craighill did his rightful share. The Episcopal Church, of which he was a life-long member, showed its appreciation by making him its deputy from the Diocese of West

Virginia to ten successive Triennial General Conventions, beginning with that of 1880, and also a delegate to the Pan-Anglican Conference in London in 1908. He was a member of the Malta Lodge of Masons, Charlestown, West Virginia, from July, 1855, to his death. In 1888 he became a member of the American Association for the Advancement of Science. He was, in 1895, the sole honorary member of the Historical Society of Maryland. In 1897 the Washington and Lee University conferred on him the degree of LL.D. The following offices were offered to him at various times and declined, viz.: Commandant of Cadets at the Virginia Military Institute in 1860; charge of the Water Department of the City of Baltimore under Mayor Malsted; Superintendent of the United States Military Academy, once by General Sheridan and once by General Schofield; President of the University of West Virginia, in 1895; Superintendent of the Coast and Geodetic Survey, by President Cleveland, and membership in the Isthmian Canal Commission, by President McKinley.

William Price Craighill was elected a Member of the American Society of Civil Engineers on October 7th, 1885; he served as a member of the Board of Direction in 1892 and 1893; and was elected President in 1894, during his term of office as a Director; he was elected an Honorary Member on March 23d, 1896.

It is probably within the truth to say that no epoch in General Craighill's career gave him more pleasure and satisfaction than his official connection with the American Society of Civil Engineers, first as a Director and finally as its President; and it certainly is a fact that this connection with the Society resulted largely to its good, and brought about many permanent and delightful friendships.

In connection with the work of the Society, he was particularly attentive to apparently trivial details, that is, when such details appeared to have an ultimate bearing of importance; on the other hand, when no subsequent advantage was apparent, he was perhaps unequaled in his ability to "side-track" such matters, without, however, appearing brusque or offensive in manner. As is frequently the case where routine matters must be passed upon by a board composed of many men, the tendency is to fritter away time to no useful purpose. Early in the General's service as a Director it was made evident that he had little in common with such discussions, and could confidently be counted on to "vote first and talk afterward." Therefore, as might be expected, he was a business executive of the first order, and it was a delight to sit with him in the Board meetings of the Society.

As a presiding officer, he was exceedingly efficient, combining in a rare degree, seldom equaled, affability, tolerance, tact, dignity, retentiveness of memory, and a swift comprehension of a controverted point. A decision never appeared to be made too quickly; nor was there any uncertainty when rendered.

It is believed that no officer of the Society made, in so few years, so many friends among the younger members of the Society; for, to the young engineer, his courteous and kindly attention was equally a delight to the recipient and to the observer.

He was in all respects well-rounded—a man of many parts. Although bound by the traditions of many years of military service, he was yet peculiarly open-minded in recognizing the necessary differences which have existed, and must exist, in the performance of engineering work under private or individual control and Governmental control. He undoubtedly possessed the attributes essential to success in any walk of life, wherein he would as surely have made his mark as he did in that branch of the Engineering Profession which he adorned so conspicuously.

General Craighill was a member of various committees of the Society, and it may serve to illustrate both the simplicity of his character and his keen discernment of a useful purpose to mention his connection with the Committee which designed the present Society badge. Here, on the face of the matter, the duty was of merely nominal importance, yet he devoted much thought to the subject, and made many suggestions. It is well remembered how, at one of these meetings, he referred with evident gratification to his West Point class pin, the class of 1853, which bore the device of a mortar, with a triangular pile of balls stacked in front of it, and over all the words, "We separate for service." The point he was making was that he believed this sentiment, in his own case at least, had proved to be a potentiality for good, and consequently the same might result from the work in hand. After the present plain lettered badge had been adopted by the Society, he took the keenest interest in having the decision of the Committee carried out, that it should be produced as nearly mechanically perfect as would be possible by available processes and manual skill.

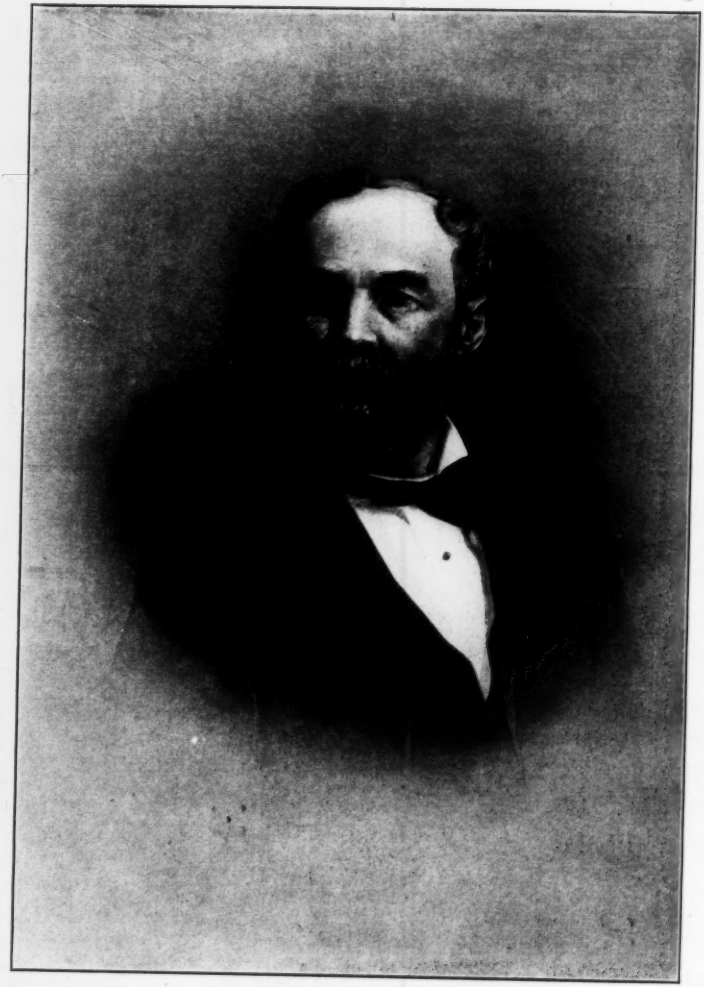
Thus it can verily be believed that in all matters, great or small, his heart was bound in the Society to the end that its influence and prestige might be broadened, the position of the engineering profession be enhanced, and that we, who "separate for service," might with pride and honor carry its insignia of membership.

This brief and all too inadequate tribute to the General's association with the Society may be best closed by his own estimate of the honors it had conferred upon him, for, by his expressed instructions, the following inscription is to be cut upon his tombstone:

WILLIAM PRICE CRAIGHILL,
Brigadier-General and Chief of Engineers,
United States Army;
Honorary Member and Past-President,
American Society of Civil Engineers.

His was a true and loving heart, strong with human feeling. A fitting crown to such a life and character is enduring memory among one's friends and associates. It is certain that he himself would have preferred no honor to his memory more acceptable than this, which, and especially from the members of the Society who had the pleasure and honor of his acquaintance, is abundantly assured.

"Such graves as his are pilgrim's shrines,
Shrines to no code or creed confined."



Chas Hermany

CHARLES HERMANY, Past-President, Am. Soc. C. E.*

DIED JANUARY 18TH, 1908.

Charles Hermany was born in Lynn Township, Lehigh County, Pennsylvania, on October 9th, 1830. He was the son of Samuel and Salome Wannemacker Hermany. His ancestors were of Swiss, French, and English stock, who had emigrated to the United States as early as 1700, and settled in the southeastern part of Pennsylvania. They were agriculturists, and, with their descendants and contemporaries, developed the district from a wilderness to ideal farming land. The farm on which Mr. Hermany was born still belongs to the family, having been deeded to his ancestor, George Philip Hermany, by William Penn.

Mr. Hermany received his early education in the public schools of his home town, where he developed a decided taste for mathematics. Having determined to adopt the Profession of Civil Engineering as his life work, he entered Minerva Seminary, at Easton, Pennsylvania, and was graduated therefrom in two years. Financial conditions prevented him from continuing his collegiate education, and for three years he taught school, at the same time continuing his studies in mathematics and in his chosen profession with a determination which characterized all his work in later life.

In May, 1853, he went West and found employment in the City Engineer's Department of Cleveland, Ohio. Here he was engaged for four years, gradually advancing to the position of First Assistant to the City Engineer, the late T. R. Scowden, M. Am. Soc. C. E. When, in 1857, Mr. Scowden, as Chief Engineer of the Louisville Water Company, took charge of the construction of the water-works system in Louisville, Kentucky, Mr. Hermany went with him as his Principal Assistant. In this capacity he served during the entire construction period and until the works were completed and put in operation in October, 1860. On January 1st, 1861, Mr. Hermany was appointed Chief Engineer and Superintendent of the water-works, which position he held continuously up to the time of his death.

The growth of the city soon proved that the works, as first constructed, were entirely inadequate, and this necessitated their general enlargement. To Mr. Hermany was assigned the duty of designing and constructing a new pumping station on the river bank, which would secure an abundant supply from the river, the stages of which varied more than 40 ft. between high and low water. The treacherous condition of the ground necessitated the sinking of a pneumatic caisson, in

*Memoir prepared by the following Committee: G. H. Benzenberg, Past-President, Am. Soc. C. E., R. Montfort, and Webster Gazlay, Members, Am. Soc. C. E.

order to secure a firm foundation for the pump-house and the pumping engine, as well as to provide for an inlet chamber. The contract was let to an experienced and responsible firm, which, however, finding the work of a hazardous nature, refused to proceed with it. This condition obliged Mr. Hermany to prosecute the work by day labor, requiring his personal supervision and utmost skill for a number of years. In this work he was successful, as he was also in the installation of pumping machinery designed principally by himself, which, at the time of its test run, attracted general attention because its high economic efficiency exceeded all known records. In addition to a betterment of the distribution system, he designed and constructed the Crescent Hill Reservoirs. His last great work was to design and construct the filter plant, including the covered clear-water basin and the high-service pumping station, with its equipment of pumping machinery.

Being extremely careful and prudent, Mr. Hermany conducted a series of exhaustive experiments, lasting more than a year, in order to determine the best method of treating the river water in the process of proposed purification. Very particular in all matters, he practically worked out every detail of all the work which he designed and constructed, the specifications which he prepared for his large pumping engine being an evidence of the minutiæ to which he carried his work. As exacting as he was in these matters, so was he in his requirement of the fulfilment of every obligation on the part of contractors. Faithful to every duty, he insisted upon the same compliance from everyone else.

As an engineer Mr. Hermany was resourceful in every emergency, which enabled him to meet successfully any contingency, as he often had occasion to demonstrate, perhaps at no time so signally as when he promptly restored the works to service by converting the remnants of a tall stand-pipe into a great air chamber, after it had been demolished by a disastrous tornado which swept over Louisville in 1890.

These qualities, as well as his skill, attained for him an eminence in his profession which resulted in his counsel and advice being sought by the water departments of many cities throughout the country. He designed the water-works at Frankfort, and at Bowling Green, Kentucky, and the pumping station at Evansville, Indiana. He also served, in different capacities, the water-works of Nashville, Memphis, St. Louis, Kansas City, Milwaukee, Providence, Chicago, and, finally, as Consulting Engineer during the construction of the new water-works of Cincinnati, Ohio.

Mr. Hermany possessed administrative abilities to a very high degree, which he manifested in conducting the office of Superintendent of Water-Works for more than forty-seven years to the perfect satisfaction of the Water Board and of every water taker, the confidence and regard of all of whom he always possessed.

His reputation for honesty and integrity was ever proverbial, and though, at times, he was impatient and blunt when he suspected unfairness or deceit—for he hated all sham and sordidness—he was at heart scrupulously just, as well as kind, generous, and thoughtful. Courteous and affable to everyone, he was deeply true and loyal to those who were privileged to be enumerated among his friends. The great affection and high esteem in which he was held by the members of his profession, his fellow-citizens, and his neighbors was shown by the great number who were present at the last sad rites, by the immense number of floral tributes under which the casket was buried, and in the last words which were spoken over his remains:

"To discover one's talents, to find one's place in the economy of the world, to put one's powers to the highest use in the service of one's fellow-men, is to merit the approbation of the Judge of all. This is what Mr. Charles Hermany did. He served this community with a faithfulness and devotion to duty that it would be difficult to equal and impossible to excel."

On December 19th, 1864, Mr. Hermany married Miss Sallie Adams, only daughter of Daniel and Sophia (Perdue) Adams. Forty-three years of happy life followed this union, during which seven children blessed their home, three of whom, Irene H., the wife of Mr. Owen Gathright, of Louisville, Emily H., the wife of Mr. James P. Donahue, of Davenport, Iowa, and Madeline, an unmarried daughter, together with his wife, survive him. During the last nine years Mr. Hermany lost his three sons and his youngest daughter, Hetty May, wife of Mr. James L. Smyser, of Louisville. These losses were severe blows to him, from the effects of which he never fully recovered.

Mr. Hermany's health was generally good, and he was ill for only a few days before his death. It had been his intention to attend the Annual Meeting of the Society in January, 1908, but, a few days before, he contracted a bad cold which rapidly developed into pneumonia, involved both his lungs, and resulted in his death within a very few hours, he having been confined to his home less than three days and to his bed less than one day.

A sad feature in connection with his death was the fact that, after many years of earnest and perplexing labor, he was about to complete the filtration works, and this was to round out his professional labors after half a century of faithful, zealous, and devoted services to his home city.

Mr. Hermany was a remarkably self-reliant man, and always exhibited a consciousness of faith in his judgment when once he had arrived at a conclusion. He was a constant reader, and was endowed with splendid intellectual gifts, possessing a mind which was quick to discern and independent in its deductions.

Mr. Hermany never held any political office, though he was always

deeply interested in questions which affected the welfare of his country, state or city, and he did not hesitate to give expression to his views. He was a loyal member of the Unitarian Church of the Messiah, and, for thirty years, was a member of its Board of Trustees. He was the first President of the Engineers' and Architects' Clubs of Louisville, and was re-elected for five consecutive terms.

On January 6th, 1869, Mr. Hermany was elected a Member of the American Society of Civil Engineers. In November, 1879, he was elected and served as a member of the Board of Direction and, in 1891, as Vice-President. In 1904, he was elected President of the Society, and presided at the meetings of the International Engineering Congress, held in St. Louis during the Louisiana Purchase Exposition. During this Congress he was elected an Honorary Member of the Institution of Civil Engineers of Great Britain.

He was also a Member of the Franklin Institute, and of the American Water-Works Association, as well as of the Salmagundi and Filson Clubs, of Louisville, the leading literary organizations of that city.

CABELL BRECKINRIDGE, M. Am. Soc. C. E.*

DIED NOVEMBER 13TH, 1907.

Cabell Breckinridge was born in Louisville, Kentucky, on November 22d, 1846. He was the youngest son of the Rev. Dr. William Lewis Breckinridge and his wife, Frances Caroline Prevost. His father was for nearly twenty-five years pastor of the First Presbyterian Church of Louisville, and later was at different times President of Oakland College, Mississippi, and of Centre College, Danville, Kentucky. His paternal grandfather was John Breckinridge of Virginia, Attorney-General of the United States during a portion of President Jefferson's administration. His grandmother was Mary Hopkins Cabell of Virginia. On the maternal side he was descended from John Bartow Prevost, a Judge of the Supreme Court of Louisiana, and his wife, Frances Ann Smith, the daughter of Dr. S. S. Smith, President of Princeton College, and granddaughter of John Witherspoon, President of Princeton College and a signer of the Declaration of Independence.

During his childhood, Cabell Breckinridge attended the school of the Rev. Joseph J. Bullock, at Walnut Hill, near Lexington, Kentucky, and was afterward a student at Oakland College, Mississippi, and at Centre College, Danville, Kentucky. At the age of eighteen, he left college before graduation, and joined an engineering party which, under the direction of Colonel Brodhead, was engaged on a reconnaissance for the Louisville and Nashville Railroad in Rockcastle County, Kentucky. From this time until the end of his life, Mr. Breckinridge was engaged, with the exception of a few interruptions in his later years on account of sickness, in the practice of his profession as a civil engineer. The record of his engagements, summarized from a memorandum left by him, is as follows:

1865—Rodman on Louisville extension of the Louisville and Nashville Railroad.
1866-70—Leveler, Transitman, and Resident Engineer on the Louisville, Cincinnati, and Lexington, now a part of the Louisville and Nashville Railroad.
1870—Engaged on a line in Wisconsin, afterward the Wisconsin Central Railway.
1871-73—Resident Engineer and Assistant Division Engineer on the Chesapeake and Ohio Railway.
1874-78—Division Engineer and Expert on the Cincinnati Southern Railway.
1878-80—United States Government service: Assistant Engineer on Cascade Locks.

* Memoir prepared by Onward Bates, President, Am. Soc. C. E.

1880-83—Chief Engineer, Alabama, Great Southern Railway, now a portion of the Cincinnati, New Orleans and Texas Pacific Railway System.
1883-84—Division Engineer on the Philadelphia Branch of the Baltimore and Ohio Railroad.
1884—United States Government service: Assistant Engineer on the Red and White Rivers.
1885-86—Engaged with the Beaver Creek Coal Company.
1886—Reconnaissance for Kentucky Central Railway (Louisville and Nashville) for lines to coal fields, and survey for a bridge over the Ohio River at Cincinnati.
1887—Division Engineer, Tennessee Midland Railroad, making surveys.
1888—With Cincinnati Southern and Louisville and Nashville Railways.
1889—United States Government: Ice Harbor Lock, Marietta, Ohio.
1890-91—Division Engineer, Kentucky Central Railroad, making surveys.
1892-94—Engaged on asphalt paving for the City of Covington, Ky.
1895-96—Division Engineer, Tennessee Central Railroad.
1896—State of Tennessee: Brushy Mountain Coal Mines.
1896-97—With Louisville and Nashville Railway, inspecting masonry, rebuilding bridges, and putting in retaining walls at Benson washout.
1897—On the United States Government survey of Warrior River.
1898-1900—Engaged on surveys for coal roads.
1901—Division Engineer, Texas and Pacific Railway, on new lines.
1902—Division Engineer on Seaboard Air Line, on survey from Rockmart to Atlanta.
1903-04—Chief Engineer, Tennessee Railway, and Tennessee Development Company.
1904-06—In private practice.
1907—Engaged on electric lines.

From this record it will be observed that Mr. Breckinridge's professional labors extended over a period of more than forty years. In that time he contributed his full share in the work of developing the prosperity of this Nation during the era of its greatest advancement. In the sense of giving his life's service to the country for the improvement of its physical condition, and the building of its transportation systems, he was a patriot and good citizen. His name deserves an honored place among those civil engineers to whom the Nation owes a debt of gratitude, for lives spent in the accomplishment of the common good, without regard for personal rewards. The nature of Mr. Breckinridge's engagements, and his devotion to the work immediately in hand, stood in the way of a widely distributed acquaintance and the general reputation he would have enjoyed under other conditions. He was greatly esteemed as an engineer by his associates who were capable of judging his attainments, and their regard for his personal qualities was remarkable. The writer of this memoir remembers him in 1869, as a splendid specimen of young manhood, with every prospect of profes-

sional success. That his ultimate success may have fallen short of what was expected by his friends, may be attributed to a character which was always kind and just to others and yet failed to do justice to himself. Any criticism of that character is lost in the feeling of affectionate regard entertained for him by those who enjoyed his acquaintance. Although the writer met Mr. Breckinridge early in his career, he was never placed in intimate relations with him, and in preparing this memoir has obtained the data for it from those who were associated with Mr. Breckinridge in his professional life. Such of his associates as are now living, including those under whose direction he was employed, and those working directly under him, as well as others who had the opportunity of judging him as an Engineer and Man, have been consulted by correspondence. While all these correspondents agree in a high estimate of him as an Engineer, they seem moved to lay a greater stress upon his manly and lovable qualities. It is proper to mention these tributes to his character in a sketch of his life, and this cannot be done in a better manner than by quoting the exact words of those who were best acquainted with him:

"He was an intelligent, energetic and capable Engineer, ready for service night or day. * * * I was very fond of him, considering him an exceptionally capable and agreeable assistant."

"Large, both in mind and body, his was a most lovable nature. He was a big-hearted, brave, generous fellow, and always a courteous gentleman."

"He was a man of high integrity, a genial companion, and true friend. He was an engineer of ability and good judgment in handling work."

"His wonderful grasp of details, and minutiæ of the profession, astounded me. * * * Personally, he was a most delightful companion, and always courteous, even under extreme provocation, so much so as to completely control an engineering party by this method alone."

"Regarding his personal character, all of us who knew him can speak in the highest terms. He was a typical Southern gentleman, of the highest, educated class. The very soul of personal honor, he was notably loyal to, and ever ready to serve his friends. Quick tempered and brave to a fault, but even more just and generous."

"He was a very accomplished engineer, a man of high capacity, absolute probity, a fine sense of honor, and great devotion to duty. * * * He was courteous and hospitable to the highest degree. In his home life he was charming. I have seen him gather his wife, ill or weary, in his arms and carry her up-stairs. He was, I may add, one of the very finest looking men, face and figure, I ever met, and a fine figure on horseback."

"I served with and under Cabell Breckinridge during the construction of the New River Division of the Chesapeake and Ohio Railroad, 1870-73, and of the Cincinnati Southern Railway in 1875-78. He had the confidence and esteem of his superiors, and the respect of his associates and subordinates. He was a man of great energy and

executive ability, with the highest ideals of courtesy, honor and integrity."

"All of our people knew him and loved him. The men who worked under him had the greatest confidence in him as an engineer and loved him as a man. From a professional point of view he was in the front rank. As a man, he possessed those qualities which endeared him to every one he met. He was brave, he was gentle, and trusted his friends implicitly. He was possessed of the highest honor."

"His work has always stood bearing testimony to his ability and faithfulness as an engineer. As to his personality, I have never known a more generous, loving and lovable nature; true and ever loyal to his friends; kind and courteous and thoughtful for strangers and those who worked for him. He made friends wherever his work called him. With a tender love of the beautiful, both in Nature and Art, he was in thorough sympathy with the artistic temperament of his gifted wife."

"He was an engineer of no inconsiderable accomplishments, an excellent organizer, an indefatigable worker, and a man of much force and vigor. As a man and gentleman, he had but few peers, and no superiors. A man of distinguished appearance, of fine presence, of courtly manners and of natural dignity, he was as much respected for the high class and order of his characteristics as he was beloved for the tenderness, gentleness, and sincerity of his nature. No man for whom I have ever worked commanded more of my respect or gained a stronger hold upon my personal affections."

If the estimate of Mr. Breckinridge's character, which may be formed from the testimony of his friends as herein stated, had been given in the words of his biographer, the latter might have been suspected of using his office to eulogize a friend, and he is grateful that he is able to use the words of others who were in a position to speak with knowledge.

Mr. Breckinridge was happily married to Miss Julia Marshall, of a distinguished Southern family. He is survived by his widow, a daughter and a son, the latter a banker in New Orleans.

Cabell Breckinridge was elected a Member of the American Society of Civil Engineers on June 1st, 1881. He died at the age of sixty-one at the Presbyterian Hospital in Atlanta, Georgia, of Bright's disease, and his body was interred at Frankfort, Kentucky.

WALTER LORTON DUSENBERRY, Jun. Am. Soc. C. E.*

DIED MAY 13TH, 1909.

Walter Lorton Dusenberry, the son of Edmund and Caroline Elizabeth (Bennett) Dusenberry, was born in New York City, on February 9th, 1862. On his father's side Mr. Dusenberry was descended from the Von Doesenbury family, who settled in New Amsterdam early in the seventeenth century. Members of the family served with distinction in the Revolutionary War. His mother's family was also of Dutch and Huguenot descent, having settled on Long Island two centuries ago.

Mr. Dusenberry received his early education at Anthon's Grammar School, in New York City. Afterward, he entered the School of Mines of Columbia University, from which he was graduated in 1884 with the degree of E. M.

For some time after his graduation, Mr. Dusenberry was connected with the United States Coast and Geodetic Survey, and was also engaged in mining engineering in the western part of the United States and in Mexico. In 1887 he received an appointment on the New Croton Aqueduct, in New York City, being promoted later for valuable services rendered. From 1889 to 1895 he served as Assistant Engineer in the Department of Public Parks. From 1895 to June, 1897, Mr. Dusenberry was employed, except for a brief interval, in the Department of Public Parks and the Department of Highways, of New York City, in charge of the opening and paving of streets. He was then appointed Engineer to the Commissioners of Accounts, and on leaving that office in 1903, he engaged in expert mining engineering work, and in the examination of properties in the Southwest and in Mexico.

Mr. Dusenberry was married on May 11th, 1899, to Nina Jay Wright, who survives him. He was a member of the Huguenot Society and the Sons of the Revolution, and was also a Mason. His personal character was of the highest, and he was devoted to his profession.

Mr. Dusenberry was elected a Junior of the American Society of Civil Engineers on November 5th, 1890.

* Memoir prepared by the Secretary from material furnished by Mr. Dusenberry's family.

TRANSACTIONS

OF THE

American Society of Civil Engineers

INDEX

VOLUME LXV

DECEMBER, 1909

SUBJECT INDEX, PAGE 536
AUTHOR INDEX, PAGE 539

Titles of papers are in quotation marks when given with the author's name.

VOLUME LXV

SUBJECT INDEX

ARCHES.

"Computation of Stresses in Open-Webbed — Without Hinges." C. W. Hudson. 145.

"Walnut Lane Bridge, Philadelphia." George S. Webster and Henry H. Quimby. 423.

BRIDGES.

"Tests of Built-Up Steel and Wrought-Iron Compression Pieces." Arthur N. Talbot and Herbert F. Moore. (With Discussion.) 202.

"The Sixth Street Viaduct, Kansas City." E. E. Howard. (With Discussion.) 42.

"Walnut Lane Bridge, Philadelphia." George S. Webster and Henry H. Quimby. 423.

BUILDINGS.

"Fire-Resistant Construction of —." An Informal Discussion. Herbert M. Wilson and others. 274.

CAISSONS.

"Caisson Disease and Its Prevention." Henry Japp. (With Discussion.) 1.

COLUMNS.

"Tests of Built-Up Steel and Wrought-Iron Compression Pieces." Arthur N. Talbot and Herbert F. Moore. 202. Discussion: John C. Moses, 246.

COMPRESSED AIR.

"Caisson Disease and Its Prevention." Henry Japp. (With Discussion.) 1.

CONCRETE.

"—Piles." Howard J. Cole. (With Discussion.) 467.

"Impurities in Sand for —." An Informal Discussion. Sanford E. Thompson and others. 250.

"Walnut Lane Bridge, Philadelphia." George S. Webster and Henry H. Quimby. 423.

COPYRIGHT.

"— in Drawings of a Technical Character." D. A. Usina. 38.

DISEASE.

"Caisson — and Its Prevention." Henry Japp. 1. Discussion: J. S. Haldane, Frederick L. Keays, Albert J. Loomis, William L. Saunders, Walton I. Aims, Seward Erdman, and T. Kennard Thomson, 24.

DRAINAGE.

"The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem." C. E. Grunsky. (With Discussion.) 294.

DRAWINGS.

"Copyright in — of a Technical Character." D. A. Usina. 38.

ELECTRIC POWER.

"Hydro-Electric Power in Canada." Cecil B. Smith. (With Discussion.) 154.

FIRE PROOFING.

"Fire-Resistant Construction of Buildings." An Informal Discussion. Herbert M. Wilson and others. 274.

FIRE PROTECTION.

"Fire-Resistant Construction of Buildings." An Informal Discussion. Herbert M. Wilson, R. W. Lesley, Richard L. Humphrey, and S. Whinery, 274.

MEMOIRS OF DECEASED MEMBERS.

Adgate, George. 514.
Breckinridge, Cabell. 529.
Craighill, William Price. 517.
Dusenberry, Walter Lorton. 533.
Hermany, Charles. 525.

MORTARS.

"Impurities in Sand for Concrete." An Informal Discussion. Sanford E. Thompson and others. 250.

PILES.

"Concrete —." Howard J. Cole. 467. Discussion: Morton L. Tower, L. J. Mensch, Charles H. Higgins, Maxwell M. Upson, Thomas C. Desmond, R. D. Coombs, Homer A. Reid, and Hunley Abbott, 488.

RAILROADS.

"Some Extensive Railroad Surveys, and Their Cost per Mile." W. S. McFetridge. (With Discussion.) 105.

RAINFALL.

— in San Francisco. 310.
— in the Philippine Islands. 388.

ROADS.

"Has Experience Demonstrated That the Oiling of — is the Most Satisfactory or Economical Method of Preventing Dust and Preserving the Road Surface?" An Informal Discussion. S. Whinery, and G. S. Davison, 462.

SAND.

"Impurities in — for Concrete." An Informal Discussion. Sanford E. Thompson, R. W. Lesley, C. P. Howard, Richard L. Humphrey, G. S. Davison, S. Whinery, Charles M. Mills, T. F. Richardson, and E. G. Haines, 250.

SEWERAGE.

"The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem." C. E. Grunsky. 294. Discussion: W. C. Hammatt, Robert G. Dieck, Charles E. Gregory, E. Kuichling, Kenneth Allen, and Walter N. Frickstad, 385.

STEEL.

"Tests of Built-Up — and Wrought-Iron Compression Pieces." Arthur N. Talbot and Herbert F. Moore. (With Discussion.) 202.

SURVEYING.

"Some Extensive Railroad Surveys, and Their Cost per Mile." W. S. McFetridge. 105. Discussion: Clinton S. Bissell, F. Lavis, E. W. Lewis, and George L. Dillman, 131.

TUNNELS.

"Caisson Disease and Its Prevention." Henry Japp. (With Discussion.) 1.

VIADUCTS.

"The Sixth Street Viaduct, Kansas City." E. E. Howard. 42. Discussion: Daniel Bontecou, Victor H. Cochrane, O. E. Mogensen, N. T. Blackburn, and George H. Pegram, 95.

WATER POWER.

"Hydro-Electric Power in Canada." Cecil B. Smith. 154. Discussion: H. Holgate, and E. J. Beugler, 194.

WROUGHT IRON.

"Tests of Built-Up Steel and — Compression Pieces." Arthur N. Talbot and Herbert F. Moore. (With Discussion.) 202.

AUTHOR INDEX

ABBOTT, HUNLEY.
Concrete piles. 507.

ADGATE, GEORGE.
Memoir of. 514.

AIMS, WALTON I.
Caisson disease and its prevention. 32.

ALLEN, KENNETH.
The sewer system of San Francisco. 400.

BEUGLER, E. J.
Hydro-electric power in Canada. 199.

BISSELL, CLINTON S.
Railroad surveys. 131.

BLACKBURN, N. T.
Sixth Street Viaduct, Kansas City. 100.

BONTECOU, DANIEL.
Sixth Street Viaduct, Kansas City. 95.

BRECKINRIDGE, CABELL.
Memoir of. 529.

COCHRANE, VICTOR H.
Sixth Street Viaduct, Kansas City. 96.

COLE, HOWARD J.
"Concrete Piles." 467.

COOMBS, R. D.
Concrete piles. 502.

CRAIGHILL, WILLIAM PRICE.
Memoir of. 517.

DAVISON, G. S.
Impurities in sand for concrete. 262.
Oiling of roads. 466.

DESMOND, THOMAS C.
Concrete piles. 498.

DIECK, ROBERT G.
The sewer system of San Francisco. 385.

DILLMAN, GEORGE L.
Railroad surveys. 143.

DUSENBERRY, WALTER LORTON.
Memoir of. 533.

ERDMAN, SEWARD.
Caisson disease and its prevention. 32.

FRICKSTAD, WALTER N.
The sewer system of San Francisco. 405.

GREGORY, CHARLES E.
The sewer system of San Francisco. 390.

GRUNSKY, C. E.
"The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem." 294.

HAINES, E. G.
Impurities in sand for concrete. 267.

HALDANE, J. S.
Caisson disease and its prevention. 24.

HAMMATT, W. C.
The sewer system of San Francisco. 385.

HERMANY, CHARLES.
Memoir of. 525.

HIGGINS, CHARLES H.
Concrete piles. 491.

HOLGATE, H.
Hydro-electric power in Canada. 194.

HOWARD, C. P.
Impurities in sand for concrete. 260.

HOWARD, E. E.
"The Sixth Street Viaduct, Kansas City." 42.

HUDSON, C. W.
"Computation of Stresses in Open-Webbed Arches Without Hinges." 145.

HUMPHREY, RICHARD L.
Fire-resistant construction of buildings. 291.
Impurities in sand for concrete. 260.

JAPP, HENRY.
"Caisson Disease and Its Prevention." 1.

KEAYS, FREDERICK L.
Caisson disease and its prevention. 26.

KUICHLING, E.
The sewer system of San Francisco. 395.

LAVIS, F.
Railroad surveys. 133.

LESLEY, R. W.
Fire-resistant construction of buildings. 290.
Impurities in sand for concrete. 258.

LEWIS, E. W.
Railroad surveys. 142.

LOOMIS, ALBERT J.
Caisson disease and its prevention. 29.

McFETRIDGE, W. S.
"Some Extensive Railroad Surveys, and Their Cost Per Mile." 105.

MENSCH, L. J.
Concrete piles. 489.

MILLS, CHARLES M.
Impurities in sand for concrete. 263.

MOGENSEN, O. E.
Sixth Street Viaduct, Kansas City. 98.

MOORE, HERBERT F.
"Tests of Built-Up Steel and Wrought-Iron Compression Pieces." 202.

MOSES, JOHN C.
Tests of compression pieces. 246.

PEGRAM, GEORGE H.
Sixth Street Viaduct, Kansas City. 101.

QUIMBY, HENRY H.
"Walnut Lane Bridge, Philadelphia." 423.

REID, HOMER A.
Concrete piles. 503.

RICHARDSON, T. F.
Impurities in sand for concrete. 265.

SAUNDERS, WILLIAM L.
Caisson disease and its prevention. 30.

SMITH, CECIL B.
"Hydro-Electric Power in Canada." 154.

TALBOT, ARTHUR N.
"Tests of Built-Up Steel and Wrought-Iron Compression Pieces." 202.

THOMPSON, SANFORD E.
Impurities in sand for concrete. 250, 271.

THOMSON, T. KENNARD.
Caisson disease and its prevention. 33.

TOWER, MORTON L.
Concrete piles. 488.

UPSON, MAXWELL M.
Concrete piles. 494.

USINA, D. A.
"Copyright in Drawings of a Technical Character." 38.

WEBSTER, GEORGE S.
"Walnut Lane Bridge, Philadelphia." 423.

WHINERY, S.
Fire-resistant construction of buildings. 293.
Impurities in sand for concrete. 262.
Oiling of roads. 462.

WILSON, HERBERT M.
Fire-resistant construction of buildings. 274.